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CONTENTS

PROCEEDINGS

PART II

TECHNICAL PAPERS

Metals

PAGE

The Tensile Properties of Metals at High Temperatures—T. D. Lynch, N. L. Mochel and P. G. McVetty 5
Typical Static and Fatigue Tests on Steel at Elevated Temperatures—T. McLean Jasper 27
Discussion on Properties of Metals at Elevated Temperatures 33
Fatigue of Metals by Direct Stress—Paul L. Irwin 53
Discussion 64
Some Fatigue Tests on Non-Ferrous Metals—R. R. Moore 66
Discussion 84
Softening of Hard-Rolled Electrolytic Copper—Norman B. Pilling and George P. Halliwell 97
A Note on the Microstructure of Aluminum-Iron Alloys of High Purity—E. H. Dix, Jr 120
Discussion 130
Accelerated Corrosion Tests on Bare Overhead Electrical Conductors—Frank F. Fowle 137
Discussion 153
The Evaluation of Corrosion Tests—E. Blough 156
Discussion 161

Cement, Concrete, Gypsum and Brick

Requirements of Cement for Modern Highway Construction—A. T. Goldbeck 169
Discussion 177
A Study of Temperatures in High-Alumina Cement and Methods of Curing—H. S. Mattimore 192
Discussion 198
A New Test for Consistency of Paving Concrete—F. H. Jackson and George Werner 204
Discussion 213
The Significance of the Common Test Methods for Determining the Strength of Mortars—John W. Gowen and H. Walter Leavitt 218
Discussion 228
Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete—Harrison F. Gonnerman 237
Discussion 251

PAGE
Studies of Bond Between Concrete and Steel—Duff A. Abrams 256
Discussion 273
Analytical Properties of Set and Hardened Mortars—E. E. Butterfield 283
Discussion 292
The Fire Resistance of Gypsum Partitions—S. H. Ingberg 299
Brick—D. Knickerbacker Boyd 315
Discussion 326

Bituminous Materials

A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures—Prévost Hubbard and F. C. Field 335
A Stability Test for Bituminous Paving Mixtures—W. J. Emmons and B. A. Anderton 346
A Deformation Test for Asphaltic Mixtures—H. M. Milburn 356
Discussion on Stability of Bituminous Paving Mixtures 365
Quality of Oil for Surface Oiling of Earth Roads and Streets—F. L. Sperry 376
Machine for the Determination of the Pliability of Prepared Roofing and the Breaking Point of Bitumen—Charles S. Reeve and Frank W. Yeager 385
Discussion 390

Paint

Tests for Hardness, Gloss, Color and Leveling of Varnishes—A. H. Pfund 392
Discussion 403
A Recording Drying-Time Meter for Varnishes and Similar Materials—J. McE. Sanderson 407

Textiles

The Design of a Research Laboratory for a Textile Manufacturing Plant—G. B. Haven 416

Specifications

The Purchase of Materials on Specification—Dean Harvey 425
Discussion 434

Subject Index 442
Author Index 449

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PROCEEDINGS

PART II

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THE TENSILE PROPERTIES OF METALS AT HIGH TEMPERATURES

BY T. D. LYNCH,[1] N. L. MOCHEL[2] AND P. G. MCVETTY[3]

ABSTRACT

This paper gives data on the tension test characteristics of several materials, comprising medium-carbon steel, nickel steel, stainless iron, and cast manganese bronze in various conditions of heat treatment and at temperatures between normal and 500° C. In addition to the results of the usual short-time tensile tests, data are presented for tests in which the stress and temperature are maintained constant over long periods of time. The effects of these sustained loads at high temperatures are shown by subsequent tension tests. Most of these data have not been previously published and they will be of value to designers.

In the short-time tests, particular attention has been paid to the accurate determination of proportional limits, as these appear to have a definite physical meaning at high temperatures. Stress-strain curves to a large scale show that the proportional limit and modulus of elasticity at high temperatures may be determined with a fair degree of accuracy.

Long-time tests show that a medium-carbon steel at 400° C. under a stress equivalent to its yield point continues to stretch over a period of about ten days with a resulting increase in tensile strength and decrease in ductility. Similar tests just above the proportional limit show continued stretch over a period of fifty days, while just below the proportional limit no stretch can be detected between the sixth and the fortieth day. These tests indicate that the propor-

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tional limit of steel as determined in the short-time tension test is the critical stress below which creep is not to be expected. Preliminary tests of non-ferrous metals indicate that this critical stress is considerably below the proportional limit.

INTRODUCTION

The use of machinery under high-temperature conditions presents a relatively new problem in machine design and it is necessary for the designer to know the effects of such temperature upon the materials used. Not only must he know the effects of high tempera-

TABLE I.—ANALYSIS AND HEAT-TREATMENT OF STEELS.

Material	Description	Chemical Composition, per cent							
		C	Mn	P	S	Si	Ni	Cr	Cu
A	Medium Carbon Steel—As rolled	0.37	0.63	0.012	0.037	0.11	.....	.....	.....
B	Medium Carbon Steel—Bars, 2 in. in diameter; normalized at 875° C.; soaked 3 hours at temperature; air cooled	0.37	0.63	0.012	0.037	0.11	.....	.....	.....
C	Medium Carbon Steel—Bars, 2 in. in diameter; quenched in water from 850° C.; drawn at 600° C.; cooled in the furnace	0.37	0.63	0.012	0.037	0.11	.....	.....	.....
D	5-per-cent Nickel Steel—Rolled bars, 1 in. in diameter; normalized at 815° C.; air cooled	0.09	0.41	0.012	0.024	0.120	4.60	0.11	.....
E	5-per-cent Nickel Steel—Forged bars, $\frac{7}{8}$ by $\frac{7}{8}$ in.; oil quenched from 785°C.; drawn at 620° C.; cooled from draw in oil	0.10	0.32	0.010	0.027	0.127	4.57	0.07	.....
F	Stainless Iron—Forged bars, $\frac{1}{8}$ by $\frac{1}{8}$ in.; oil quenched from 955°C.; drawn at 565°C.; cooled from draw in oil	0.09	0.52	0.010	0.017	0.47	0.52	12.23	nil

TABLE II.—ANALYSIS AND HEAT-TREATMENT OF CAST MANGANESE BRONZE.

Material	Description	Chemical Composition, per cent						
		Cu	Sn	Fe	Mn	Zn	Pb	Al
G	Cast Manganese Bronze—As cast	58.4	0.79	0.68	0.38	39.0	0.031	0.71

ture but also the effects of stress sustained by the material over long periods of time at high temperature.

The Research Department of the Westinghouse Electric and Manufacturing Co., in conjunction with the South Philadelphia Works, has been carrying on investigations along these lines for some considerable time in order to make possible the better determination of safe working stresses. Some data on these temperature effects have already been published[1] and this paper discusses the later developments.

[1] Wilhelm, Discussion, Symposium on Effect of Temperature Upon the Properties of Metals, *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 142 (1924).

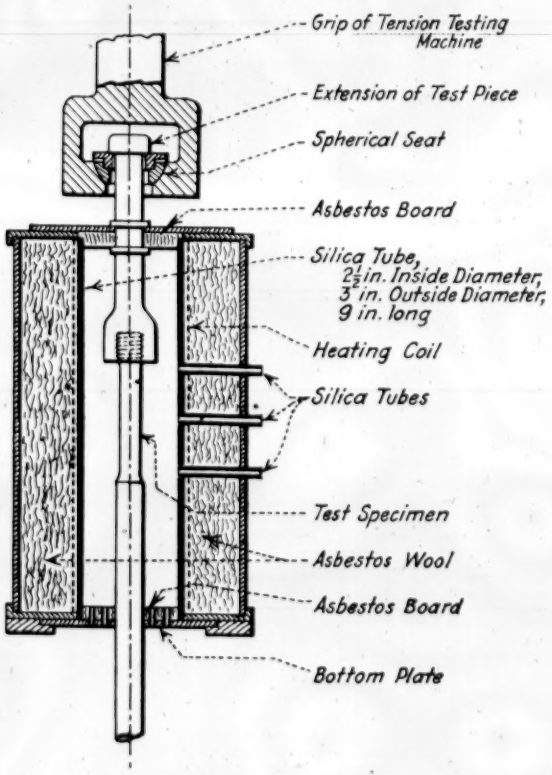


FIG. 1.—Electric Furnace for Tension Tests.

Coil wound from ¼ by 0.032-in. nichrome ribbon; resistance, 0.125 ohms per foot.

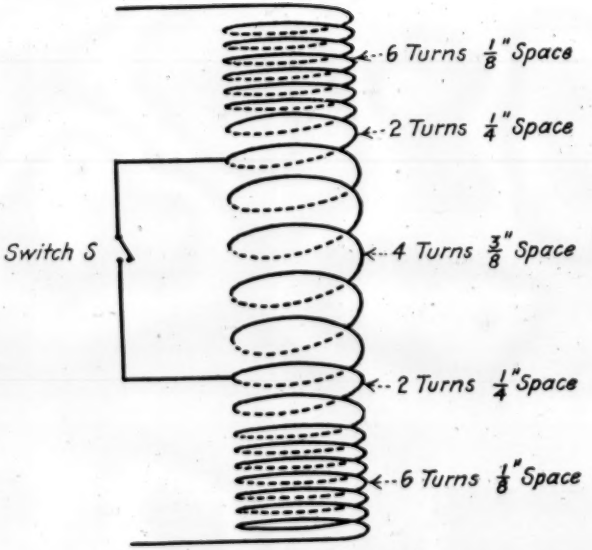


FIG. 2.—Showing Unequal Spacing of Coil to Compensate for Cooling.

MATERIALS TESTED

Table I shows the analysis and heat-treatment of the steels tested and Table II shows similar data for a representative non-ferrous metal.

SHORT-TIME TEST EQUIPMENT

An hydraulic type of tension testing machine of 100,000-lb. capacity was used in making the first series of tests in which the effects of temperature alone were determined.

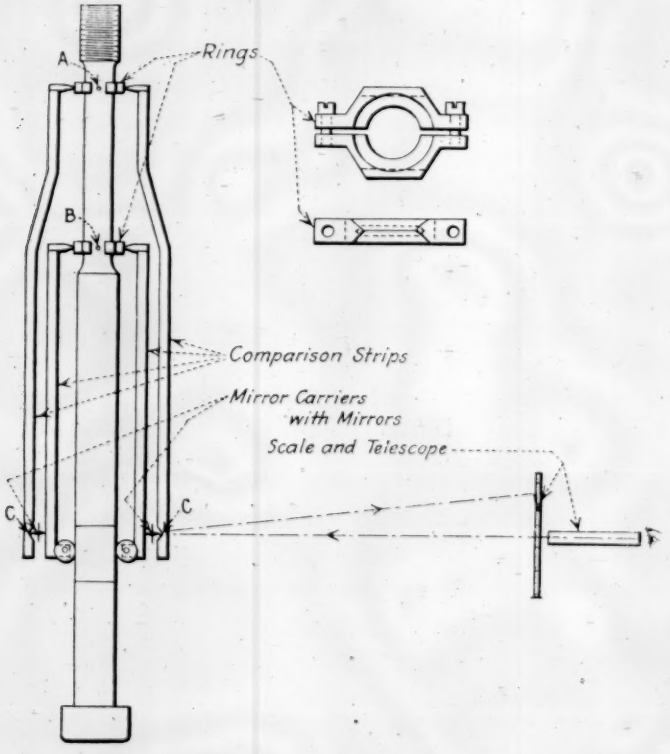


FIG. 3.—High-Temperature Extensometer.

The method of heating the test piece is essentially the same as that previously described.[1] It seems desirable, however, to give the following additional information.

The resistance type of furnace, Fig. 1, has been found to give a very satisfactory method of heating since the spacing of the turns, Fig. 2, can be so arranged that the temperature is made uniform over

[1] *Loc. cit.*

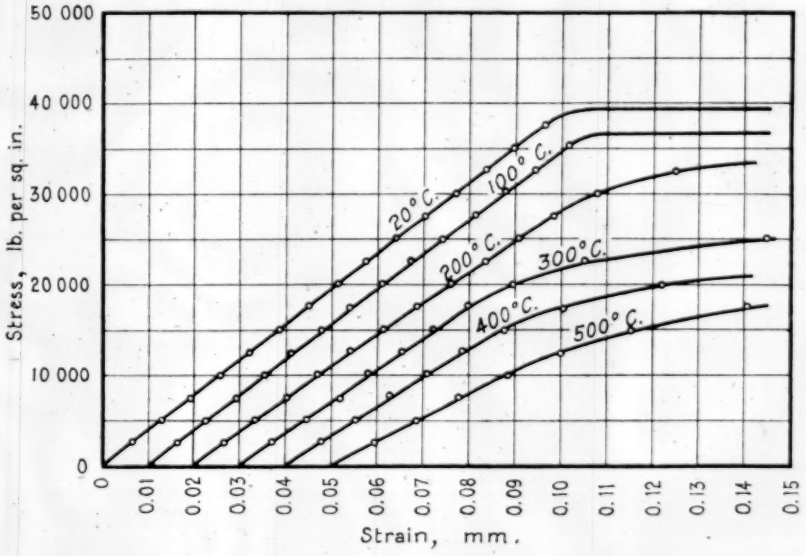
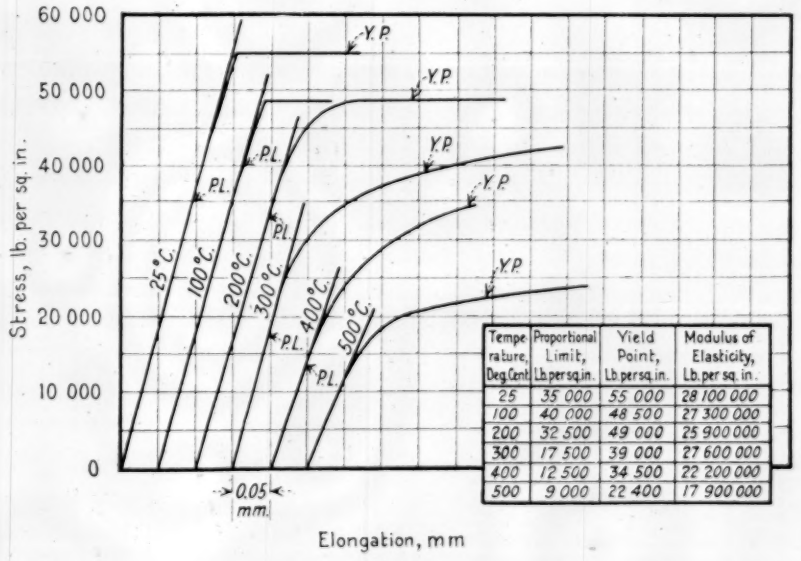
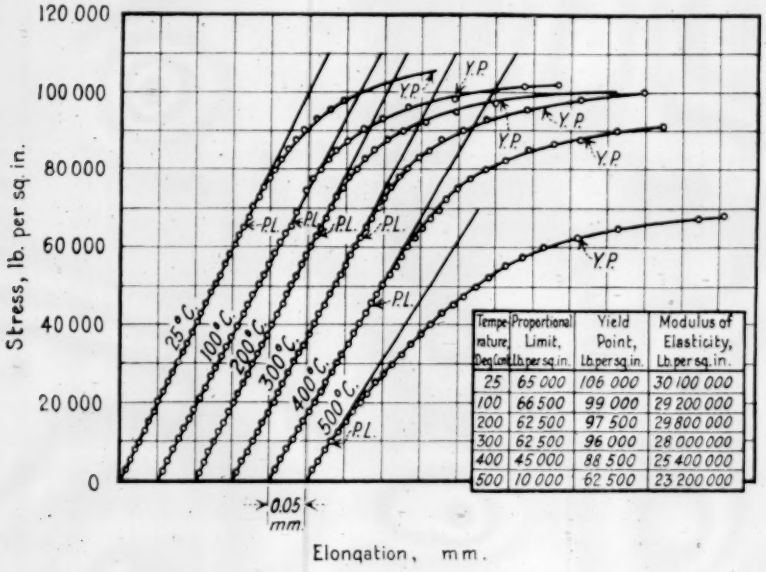


FIG. 4.—Stress-Strain Curves for Medium-Carbon Steel, Normalized (Material B) Taken at Various Temperatures.



Temperature, Deg. Cent.	Proportional Limit, Lb. per sq. in.	Yield Point, Lb. per sq. in.	Modulus of Elasticity, Lb. per sq. in.
25	35 000	55 000	28 100 000
100	40 000	48 500	27 300 000
200	32 500	49 000	25 900 000
300	17 500	39 000	27 600 000
400	12 500	34 500	22 200 000
500	9 000	22 400	17 900 000

FIG. 5.—Effect of Temperature on Elastic Properties of Rolled and Normalized 5-per-cent Nickel Steel (Material D).



Temperature, Deg. Cent.	Proportional Limit, Lb. per sq. in.	Yield Point, Lb. per sq. in.	Modulus of Elasticity, Lb. per sq. in.
25	65 000	106 000	30 100 000
100	66 500	99 000	29 200 000
200	62 500	97 500	29 800 000
300	62 500	96 000	28 000 000
400	45 000	88 500	25 400 000
500	10 000	62 500	23 200 000

FIG. 6.—Effect of Temperature on Elastic Properties of Stainless Iron, Forged and Heat Treated (Material F).

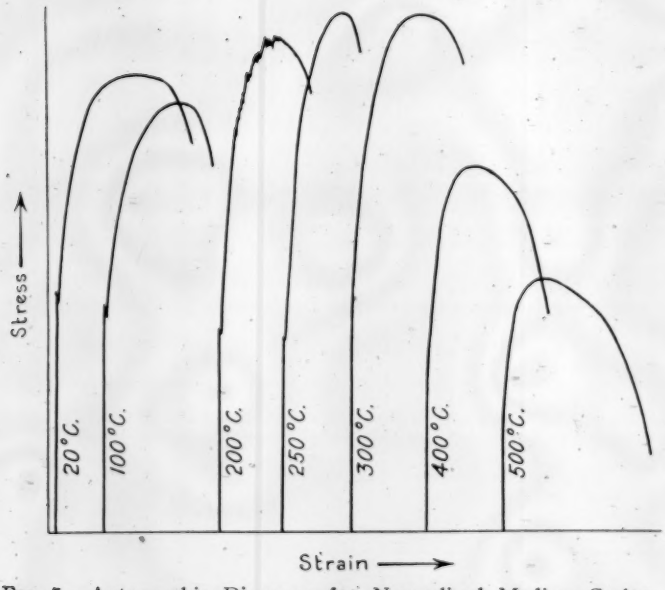


FIG. 7.—Autographic Diagrams for Normalized Medium-Carbon Steel (Material B) Tested at Various Temperatures.

the entire gage length. These furnaces require calibration in the testing machine with measurements of temperature at the center and ends of the gage length after temperature equilibrium has been reached. These data are used as a basis for changing the spacing of the winding (Fig. 2). Furnaces were thus obtained which gave a fairly uniform temperature over the gage length. In the determina-

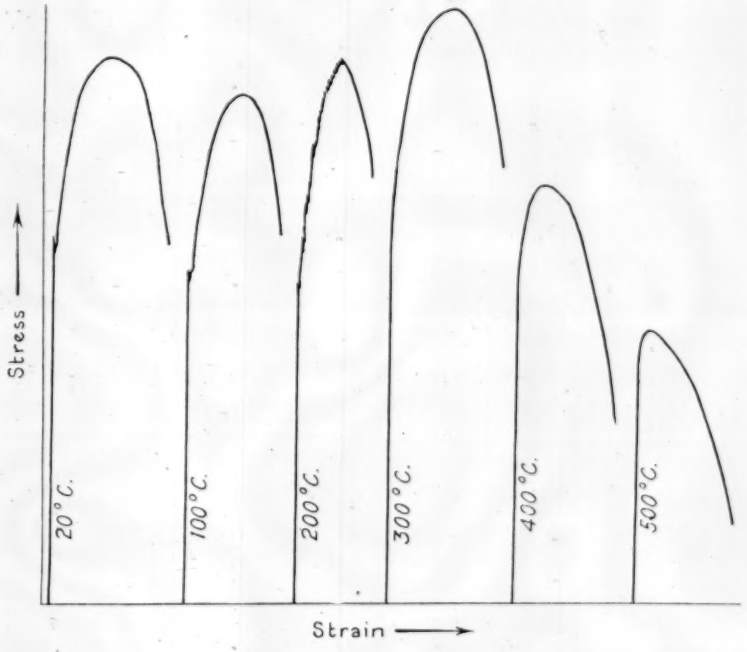


FIG. 8.—Autographic Diagrams for Heat-Treated Medium-Carbon Steel (Material C) Taken at Various Temperatures.

tion of test results subsequently discussed, the following values of temperature may be considered as representative:

Top of gage length	420° C.
Center of gage length	430° C.
Bottom of gage length	424° C.

These values show a variation of 2.3 per cent and considerable improvement is expected in furnaces now being built.

Details of the method of strain measurement as shown in Fig. 3 have been previously described.[1]

[1] *Loc. cit.*

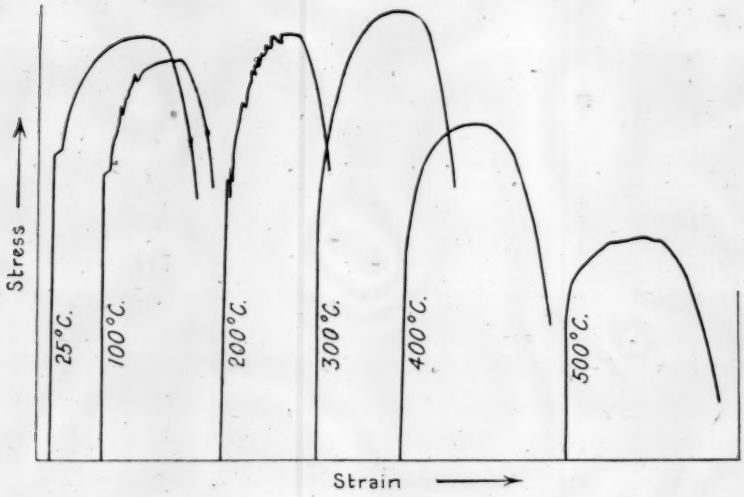


FIG. 9.—Autographic Diagrams for Rolled and Normalized 5-per-cent Nickel Steel (Material D) Tested at Various Temperatures.

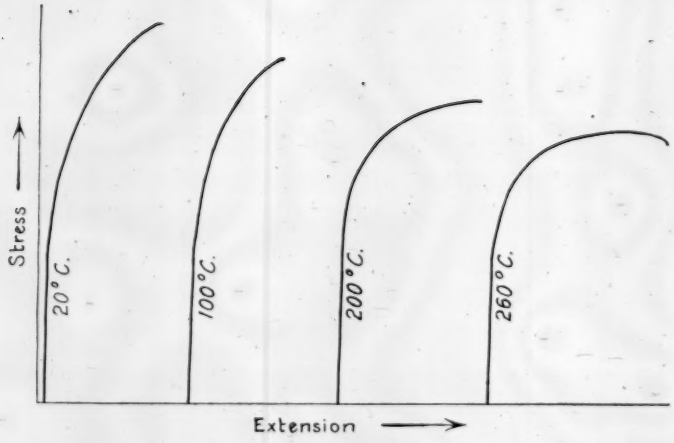


FIG. 10.—Autographic Diagrams for Manganese Bronze (Material G) Tested at Various Temperatures.

SHORT-TIME TEST RESULTS

Figs. 4, 5 and 6 show typical stress-strain diagrams for materials B, D and F in which the effect of temperature on the proportional limits is apparent. The scale of extension of Fig. 4 is enlarged to

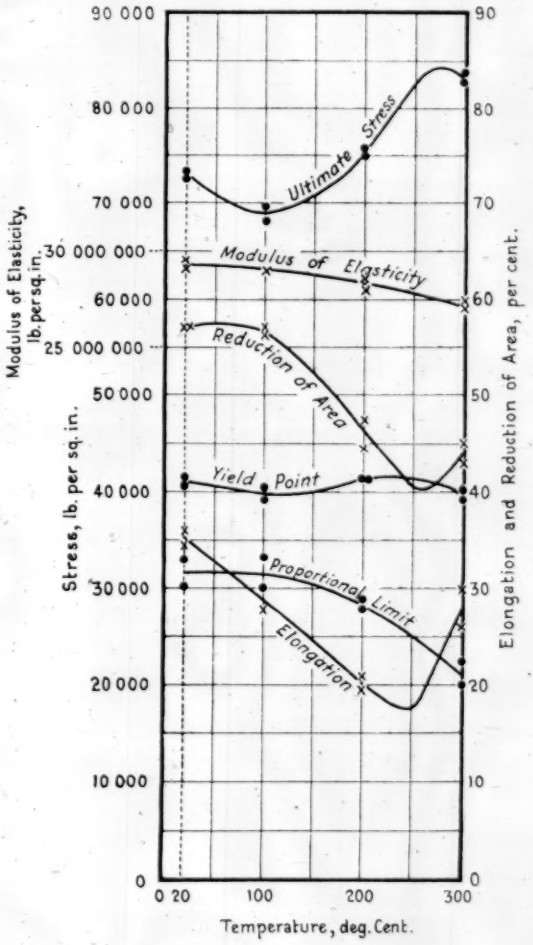


FIG. 11.—Tensile Properties at High Temperatures of Medium-Carbon Steel, as Rolled (Material A).

show the change in the slope of the curves. Since this slope is a measure of the modulus of elasticity, it is evident that the effect of temperature upon this modulus may be determined with a fair degree of accuracy.

Figs. 7, 8, 9 and 10 show similar data for the complete range of the tests. These are typical autographic diagrams drawn by the testing machine, and they show clearly the effect of temperature upon the ultimate strength and ductility.

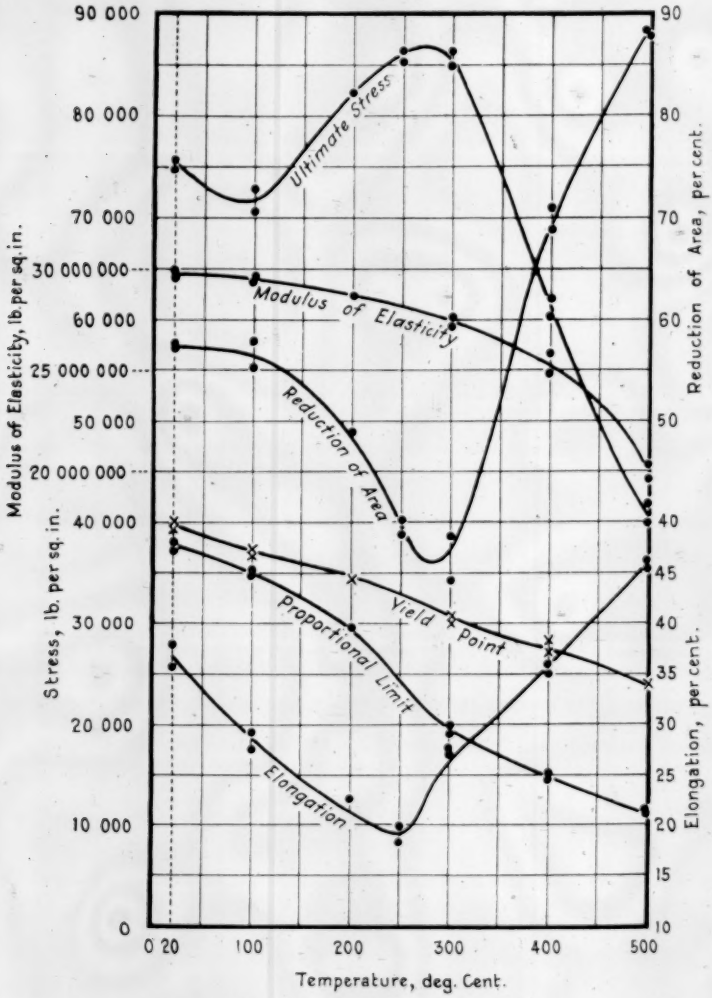


FIG. 12.—Tensile Properties at High Temperatures of Medium-Carbon Steel, Normalized (Material B).

Figs. 11, 12 and 13 show the effects of temperature upon the tensile properties of a medium-carbon steel in the rolled, normalized and heat-treated conditions, respectively (Material A, B and C). The elastic properties as measured by the proportional limit and modulus of

elasticity are reduced with the increase of temperature, while the strength decreases with the increase of temperature to about 100° C. and then increases to a maximum at about 275° C. From this point,

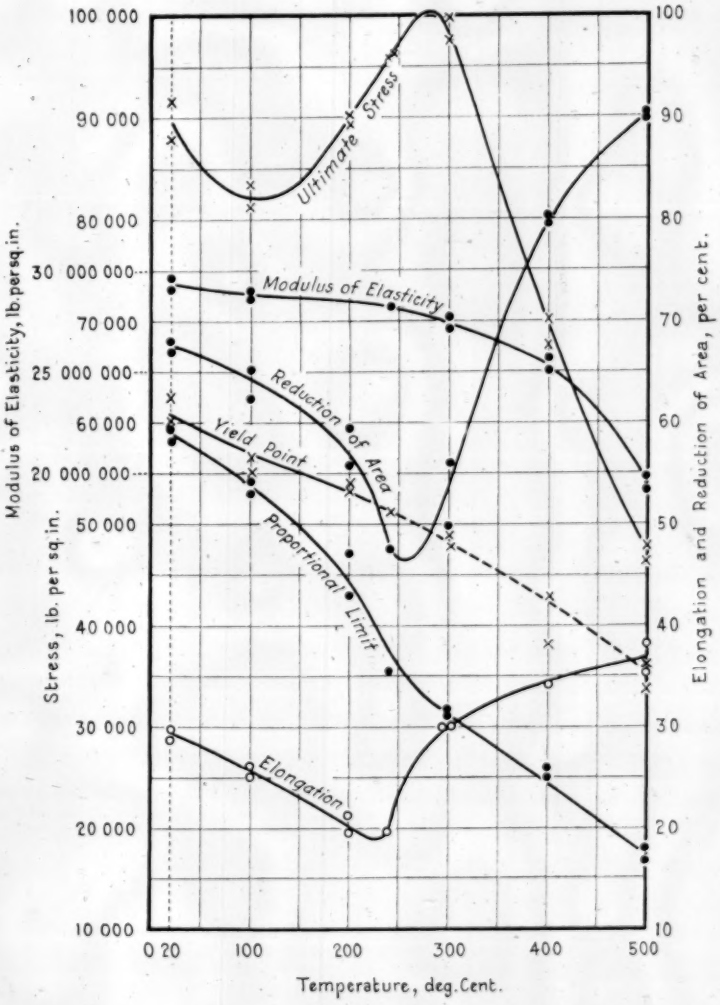


FIG. 13.—Tensile Properties at High Temperatures of Medium-Carbon Steel, Heat Treated (Material C).

any increase in temperature decreases the ultimate strength. The ductility as measured by the elongation and reduction of area decreases in general up to about 250° C. and then increases with increasing temperature.

Figs. 14 and 15 show similar data for a 5-per-cent nickel steel in the normalized condition (Material D) and in the heat-treated condition (Material E). The variation of the properties with tempera-

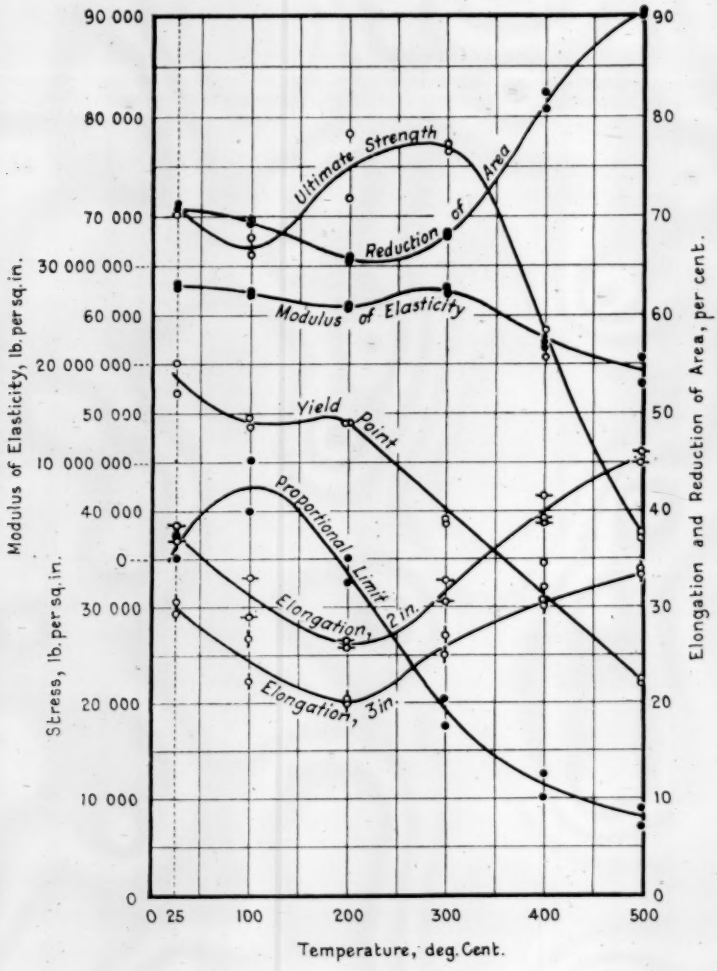


FIG. 14.—Tensile Properties at High Temperatures of Rolled and Normalized 5-per-cent Nickel Steel (Material D).

ture is somewhat similar to that previously shown for medium-carbon steel. The ductility is considerably higher, especially at its minimum value, about 250° C.

Fig. 16 shows similar data for stainless iron (Material F).[1] The properties of this material are affected in a less degree by high temperature than any of the other materials examined. The elastic

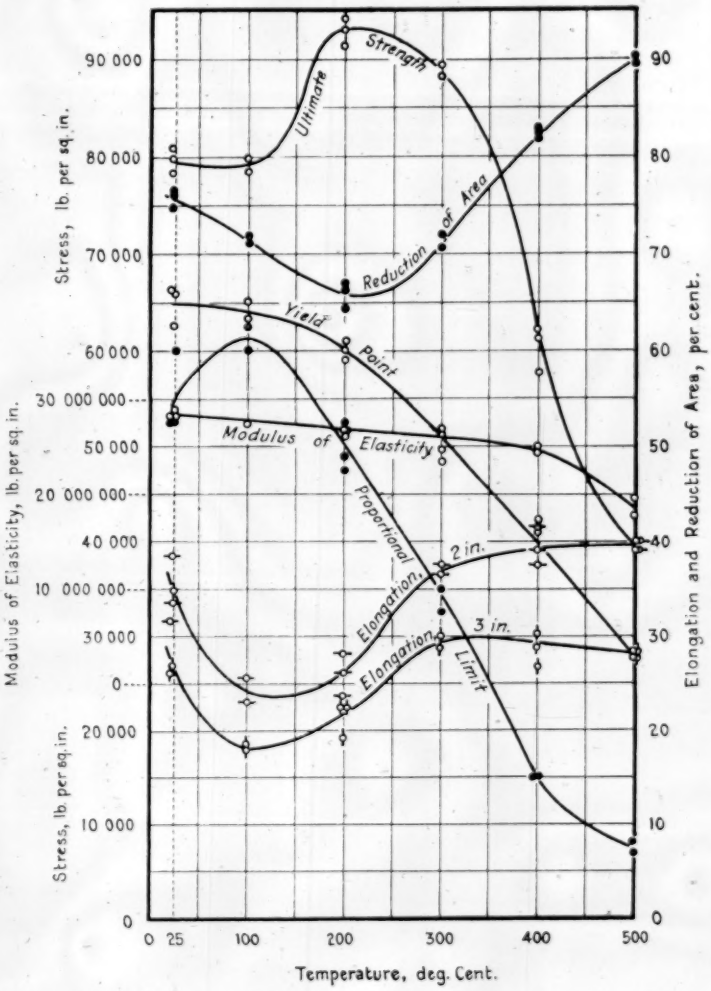


FIG. 15.—Tensile Properties at High Temperatures of Forged and Heat-Treated 5-per-cent Nickel Steel (Material E).

properties as measured by the proportional limit and modulus of elasticity decrease less than 10 per cent between 25 and 300° C.

[1] Some remarks have already been made on this material by Mr. J. M. Lessells at the May, 1925, meeting of the American Society for Steel Treating.

The ductility as measured by the reduction of area decreases less than 5 per cent between 25 and 400° C. The ultimate strength at 500° C. is about twice that of the carbon and nickel steels at the same temperature.

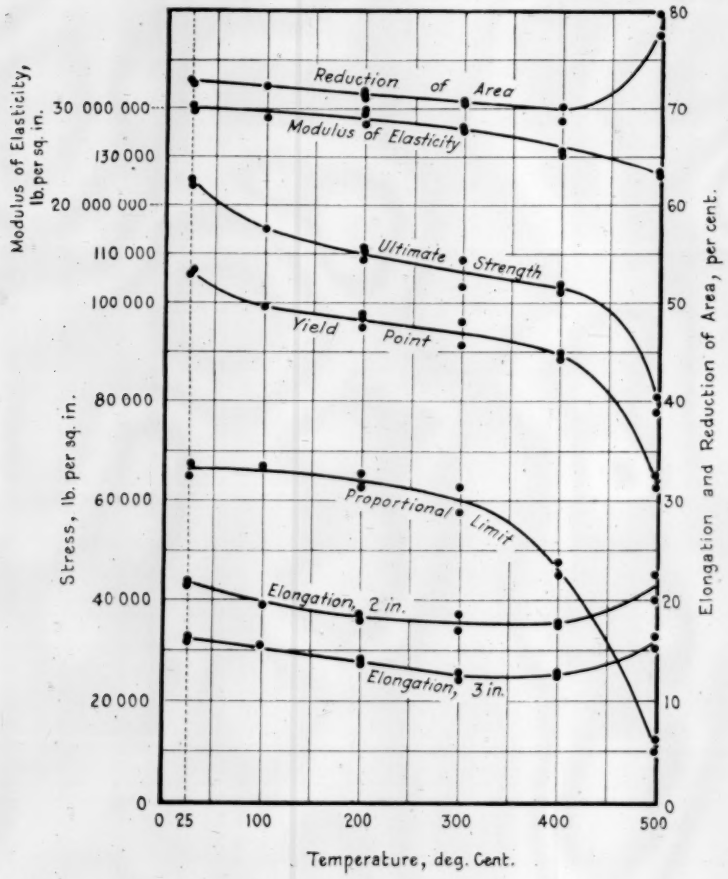


FIG. 16.—Tensile Properties at High Temperatures of Forged and Heat-Treated Stainless Iron (Material F).

Considerable work has also been done on non-ferrous metals, of which cast manganese bronze (Material G) is cited as an example.

Fig. 17 shows the tensile properties over the temperature range of 20 to 260° C. The elastic properties appear to be only slightly affected through this range of temperature, but the ultimate strength shows a considerable reduction.

LONG-TIME TEST EQUIPMENT

The second series of tests, in which the effects of continued stress at high temperatures were determined, required special equipment. This is shown in Fig. 18. The test piece is suspended from a steel frame through a ball-and-socket joint and carries at the lower end a similar ball-and-socket joint which supports a yoke carrying a knife

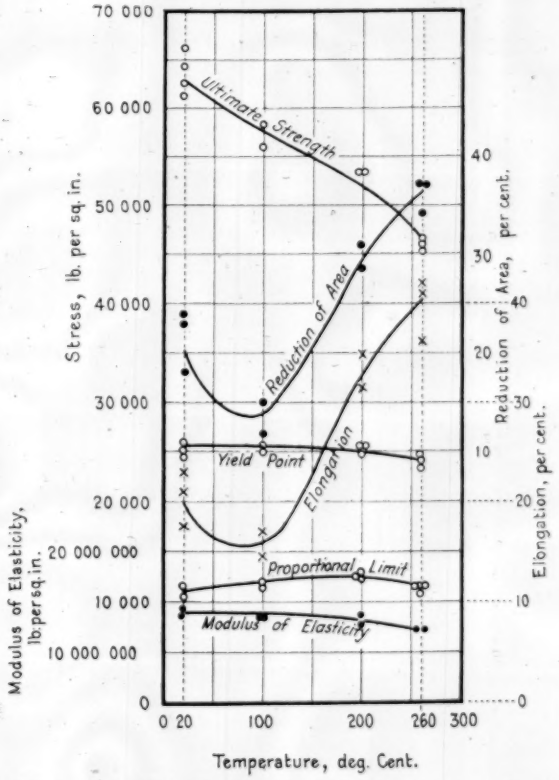


FIG. 17.—Tensile Properties at High Temperatures of Cast Manganese Bronze (Material G).

edge. A lever, pivoted to the frame at one end, is suitably located on this knife edge and can be loaded at the free end to give the desired stress. A load of 1 lb. at the end of the lever gives a load of 25 lb. on the test piece.

LONG-TIME TEST RESULTS

The results obtained when the materials were stressed for long periods of time at high temperature show the importance of this type of test.

Several tests were made on the medium-carbon steels, materials B and C, at 400° C. and at various stresses. Fig. 19 shows the effect on both materials of a stress of 25,000 lb. per sq. in. at a temperature of 400° C. In the same figure, short-time tension test curves of both materials at 400° C. are shown for reference.

The stress chosen for the comparative long-time tests represents the yield point of the normalized material, material B, and the pro-

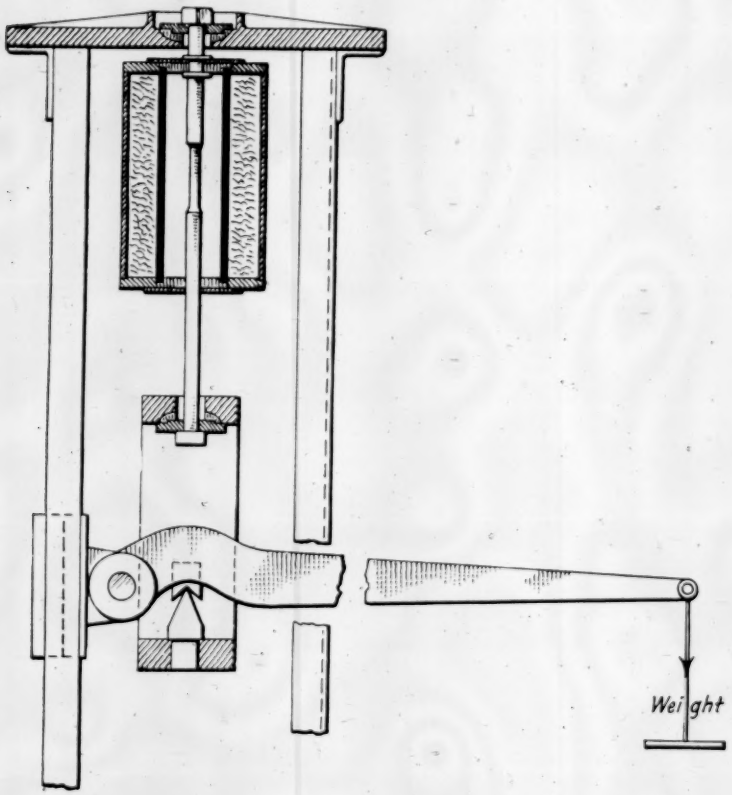


FIG. 18.—Long-Time Test Apparatus.

portional limit of the heat-treated material, material C. This stress is 67 per cent above the proportional limit of material B at 400° C. Material B shows continued extension over a period of more than fourteen days and material C shows no tendency to stop stretching within a period of fifty days. It is interesting to note that the final rate of extension of material C is about one-half that of material B.

Further tests were made on material B at 400° C. and 14,000 lb.

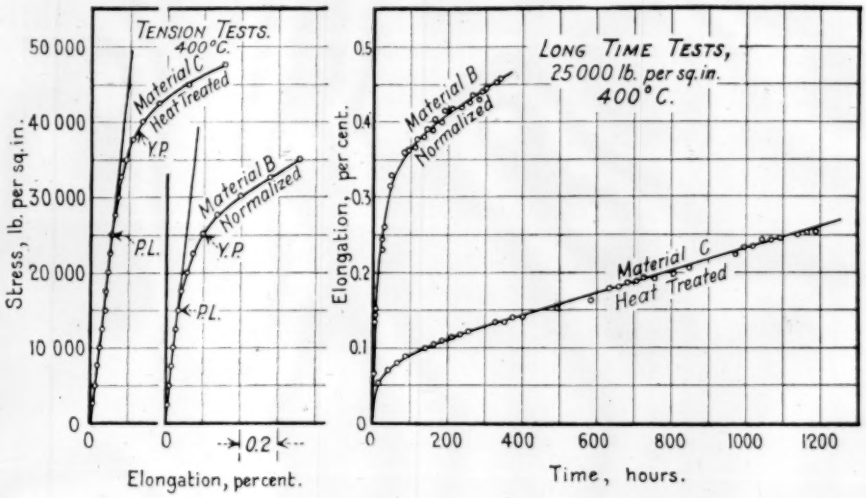


FIG. 19.—Tests on Medium-Carbon Steel.

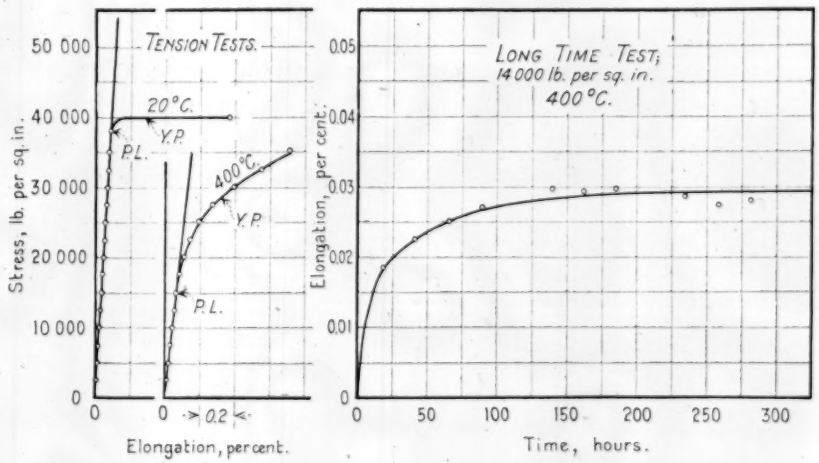


FIG. 20.—Tests on Normalized Medium-Carbon Steel.

per sq. in. This stress is 7 per cent below the proportional limit at 400° C. Fig. 20 shows the effect of time on the elongation produced. In the same figure, short-time tension test curves for this material at 20 and 400° C. are shown for reference.

The rate of stretch is rapid at first but becomes zero after about 150 hours. This test was later prolonged over 900 hours without evidence of further stretching.

Since material C, Fig. 19, shows continued stretch over a period of 50 days at the proportional limit while Fig. 20 shows that stretch stops within a week at a stress 7 per cent below the proportional limit, it appears that the proportional limit closely approximates the critical

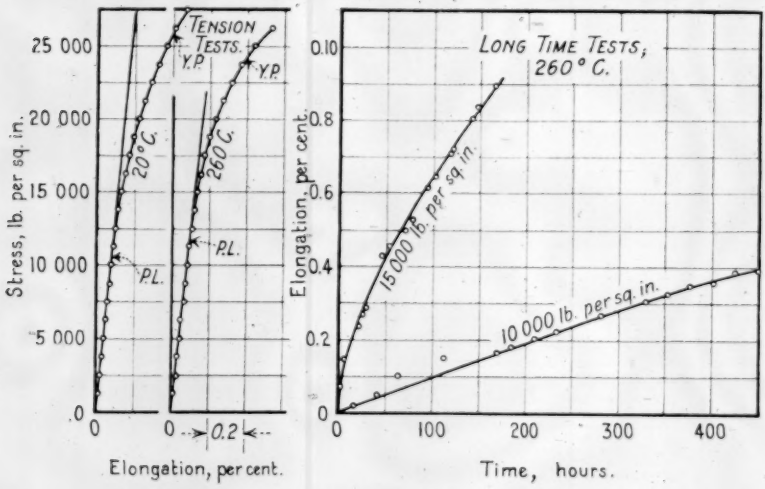


FIG. 21.—Tests on Cast Manganese Bronze.

stress. It is possible, also, that the stress used in the test on material C plotted in Fig. 19 is slightly above the proportional limit of that particular test bar. While the results so far obtained will require further confirmation, it appears that sufficient information is available to advance the conclusion that a stress corresponding to or slightly lower than the proportional limit can be carried by this material indefinitely without causing continued extension. It is interesting to note that a hypothesis recently advanced by Mr. R. W. Bailey[1] seems to agree with this conclusion.

Considerable work has also been done on non-ferrous metals of which manganese bronze, material G, may be cited again as an

[1] *Engineering*, April 24, 1925, p. 518.

example. Fig. 21 shows long-time test curves for 260° C. and stresses of 10,000 and 15,000 lb. per sq. in. These stresses are 10 per cent below and 30 per cent above the proportional limit, respectively.

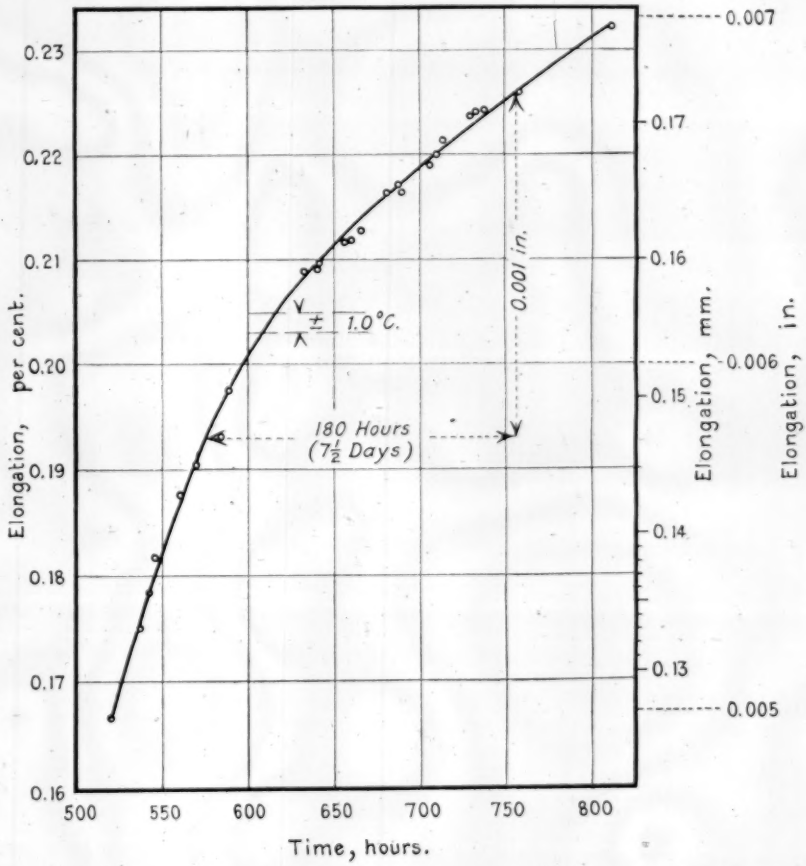


FIG. 22.—Effect of Temperature Variations in Long-Time Tests on Properties of Medium-Carbon Steel, Heat Treated (Material C), Tested at 400° C., 25,000 lb. per sq. in.

Proportional Limit at 400° C., 24,500 lb. per sq. in.

It is evident that there is no tendency for stretch to stop in either case. In the same figure, short-time tension tests of this material at 20 and 260° C. are shown for reference.

Tests on other non-ferrous metals seem to confirm these data in that they show no tendency of the material to cease stretching at high

temperatures, even when stressed considerably below the proportional limit.

Comparing the rate of stretch for manganese bronze, material G, at 260° C. under a stress 30 per cent above the proportional limit, with the rate of stretch for normalized medium-carbon steel, material B,

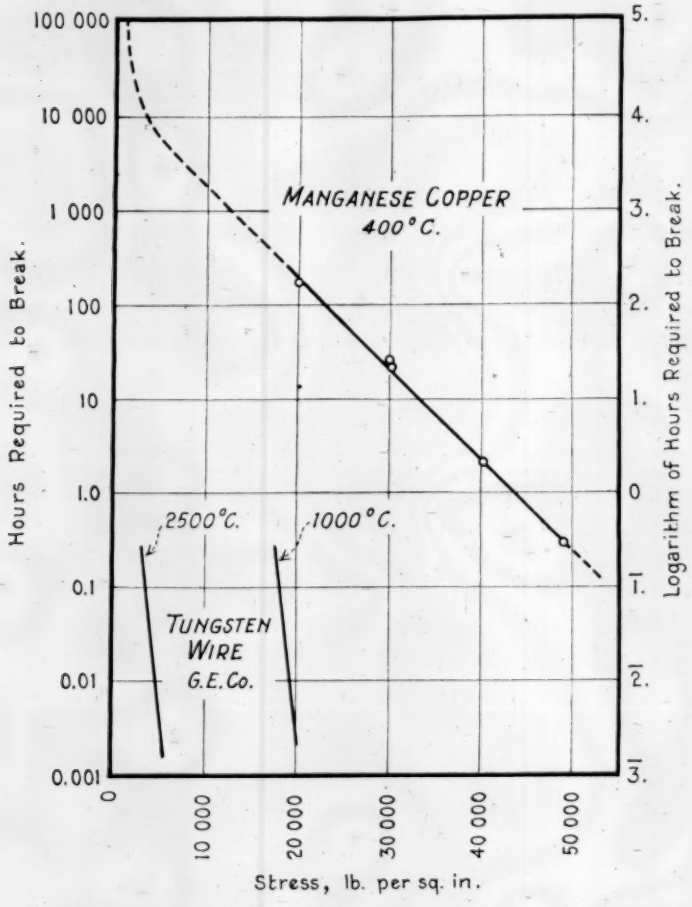


FIG. 23.—Effect of Stress on Time Required to Break in Long-Time Test at Constant Temperature.

at 400° C. under a stress 67 per cent above the proportional limit, in the period from 100 to 160 hours, it is interesting to note that the rate of stretch for the manganese bronze is more than eight times that of the steel.

In the presentation of data of this kind a word or two must be said in reference to temperature control. This must necessarily be

very close if creep phenomena are to be accurately determined. Fig. 22 indicates the degree of control maintained in these tests. This represents to an enlarged scale a portion of the curve for material C, shown in Fig. 19. The scatter of the points indicates that the temperature variation is within $\pm 1^\circ$ C.

So far, we have discussed some critical stress values below which a material will not stretch indefinitely and have shown that these agree approximately with the proportional limit for steels but are very much less for certain non-ferrous metals. It is also of interest to study the phenomena of stretching when the stresses are considerably above the proportional limit. Such stresses produce fracture if the test is sufficiently prolonged. If the stress on successive test bars is reduced, the time to cause fracture is more than proportionally increased. Plotting the stress in pounds per square inch against the logarithm of the time to cause fracture gives a straight line relation, as shown in Fig. 23 for manganese copper. In the same figure are also plotted certain results obtained by other investigators[1] on tungsten wire.

TABLE III.

LONG-TIME TESTS.—						
Mark on Test Specimen	721-14	721-16	721-17	Average		
Testing temperature, deg. Cent	400	400	400			
Stress, lb. per sq. in	25 000	25 000	25 000			
Duration of test, hours	210	230	210			
SUBSEQUENT TENSION TESTS.—						
Mark on Test Specimen	721-14	721-16	721-17	.....	721-1	721-10
Testing temperature, deg. Cent	20	20	20	.....	20	400
Proportional limit, lb. per sq. in	40 000	41 500	38 500	40 000	37 500	15 000
Yield point, lb. per sq. in	42 000	44 000	40 000	42 000	39 500	25 000
Ultimate strength, lb. per sq. in	77 100	77 500	77 200	77 266	75 600	62 000
Elongation in 2 in., per cent	35	34.5	35.7	35	38	35
Elongation in 3 in., per cent	28.6	28.6	29.3	28.8	32	27
Reduction of area, per cent	57	53	53	54.3	57	71
Modulus of elasticity, lb. per sq. in	29 900 000	29 900 000	29 800 000	29 870 000	29 800 000	25 400 000

These results, taken in conjunction with those previously discussed on metals stressed at or near the proportional limit, seem to indicate that this curve must tend rapidly upward toward the left as shown by the dotted line and that it must be asymptotic to the critical stress value. This critical stress value for non-ferrous metals appears to be very low or it may not exist. The determination of such values for different materials is part of our present program.

[1] *Philosophical Magazine*, Vol. 28, 1924, p. 243.

EFFECT OF LONG-TIME TESTS ON SUBSEQUENT SHORT-TIME TESTS

In addition to obtaining curves of stretch for different materials it is also important to know whether or not the stressing of materials for long periods of time at high temperatures has any effect on the elastic and ductility values. To determine this, three normalized medium-carbon steel bars, material B, were stressed for 210 to 230 hours at 205,00 lb. per sq. in. and 400° C. These were then cooled and short-time tension tests made at normal temperature. The results are given in Table III. Data on the unstressed material at 20 and 400° C. are included for purposes of comparison.

Stressing beyond the proportional limit at high temperature produces an increase in proportional limit, yield point and ultimate strength with a corresponding decrease in ductility. The modulus of elasticity is not appreciably affected. These effects are similar to those produced by cold working and subsequent tempering. It is felt that valuable information regarding this cold working effect will be secured by stressing a number of specimens of the same material under identical load and temperature for varying lengths of time, observing the "flow" of the metal during that time, and finally breaking at the high temperature. The results should also throw much light on the question of deterioration of material. A true comparison of results of short-time tests and of variable long-time tests will also be possible. Especial attention is being given to this feature in the continuation of this work, which is a part of the program of the Westinghouse Company.

CONCLUSIONS

Evidence is advanced relative to the importance to the designer of long-time tests. It is shown that the proportional limit of medium-carbon steel as determined in the short-time test has a very definite physical meaning at high temperatures, and that continuous creep at or below this stress is not to be expected. On the other hand, this is not true for the non-ferrous materials examined, although metallurgical changes may be shown by further work to account for such differences.

The results on the whole show the vital importance of such data to the designer so that safe and economical designs may be accomplished.

Acknowledgment.—The authors wish to acknowledge their indebtedness to Mr. J. M. Lessells, under whose supervision the experiments were conducted, and to the Westinghouse Electric and Manufacturing Co. for permission to publish the results.

[For discussion, see page 33.—ED.]

TYPICAL STATIC AND FATIGUE TESTS ON STEEL AT ELEVATED TEMPERATURES

By T. McLean Jasper[1]

Synopsis

This paper gives typical results of investigations which were carried out partly in the Fatigue of Metals Laboratory at the University of Illinois and partly in other laboratories, for the purpose of obtaining a better knowledge of some of the factors governing the static and fatigue properties of wrought ferrous metals. The sustaining effect of alloying materials on the strength of wrought ferrous metals at elevated temperatures is also indicated.

Typical short-time static experimental results for a high-strength steel at elevated temperatures are compared with long-time static experimental results of the same steel under similar temperature conditions. Typical short-time static experimental results for a low-carbon normalized steel at elevated temperatures are compared with long-time static experimental results of the same steel under similar temperature conditions, together with a discussion of the effect of heat treatment on the static values of strength.

Special attention is drawn to the fact that at temperatures above from 400 to 600° F., the long-time static test gives values of tensile strengths which are materially lower than those obtained by the test made at the ordinary laboratory speed of testing.

An explanation is attempted giving the reason for the increase of the ultimate strength at the blueing temperature of a normalized steel.

It is suggested that there is an effect of speed of testing and temperature on fatigue strength similar to that found in static values of strength.

The use of metals at elevated temperatures has become an important question to builders and users of steam generating machinery and internal combustion engines. Certain parts of such machinery are subjected to elevated temperatures, and these temperatures have steadily increased in recent years with the improvement in size and efficiency of such equipment until, at the present time, metal under considerable stress is subjected to temperatures which keep them constantly at from 600 to 900° F. Certain metal parts used at elevated temperatures, such as steam piping, valves, boilers, and turbine cases, are subjected in general to static stresses only. Other metal parts, such as turbine wheels, turbine blades, piston rods, and

[1] Special Research Assistant Professor of Engineering Materials, University of Illinois.

cylinders of internal combustion engines, are subjected to reversed stresses which may be repeated many times.

The metals in general use at elevated temperatures are almost all ferrous, and consist in the main of wrought steel, cast steel and cast iron. It is gradually becoming known that certain ferrous alloys are capable of withstanding stress at high temperatures better than others. Such metals are usually high in tungsten, nickel, chromium, or some satisfactory combination of those alloys. Fig. 1 gives a general idea of the relative values of steels containing these alloys when tensile strength at elevated temperatures is considered. These

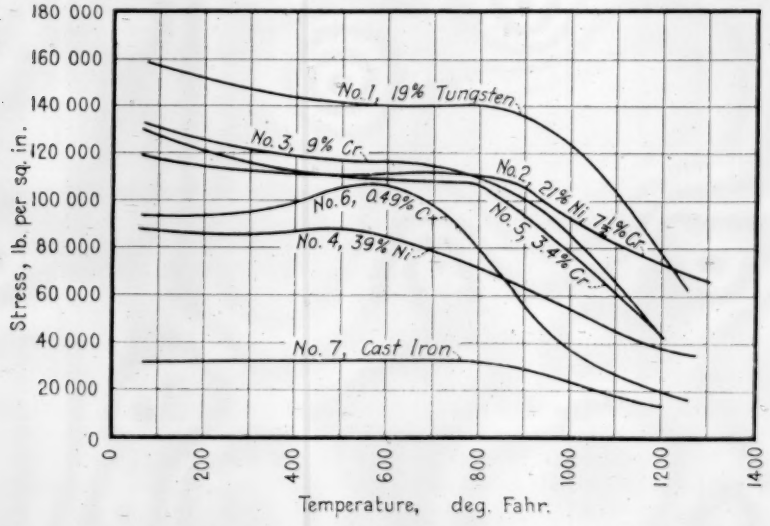


FIG. 1.—Curves Showing the Effect of Certain Ingredients on the Tensile Strength of Various Annealed Steels and Cast Iron at Elevated Temperatures.

data are from annealed steels and cast iron, and are largely drawn from results by Harper and MacPherran.[1] The results of tests on steels at elevated temperatures show that in general the static properties other than strength are affected by temperature in the following manner: the higher the strength for any particular steel, the lower the percentages of reduction of area and elongation and the higher the hardness factors.

In obtaining strength data at elevated temperatures for use in actual design, care must be exercised to approach as nearly as possible the conditions under which the metal is to be used. The method usually adopted in testing metals at ordinary temperatures has been to make a test which lasts less than ten minutes, and expect this to

[1] *Bulletin No. 141*, Allis-Chalmers Manufacturing Co., 1922.

represent the strength and ductility factors to which the metal will conform when subjected to stress for months, or even years. For wrought and cast ferrous metals at ordinary atmospheric temperatures, this assumption may be made without serious error, but at elevated temperatures an error of from thirty to fifty per cent depending on the temperature used may result when this procedure is followed. Static testing at elevated temperatures, therefore, is not so simple, nor can it be as expeditiously carried out as at ordinary atmospheric temperatures.

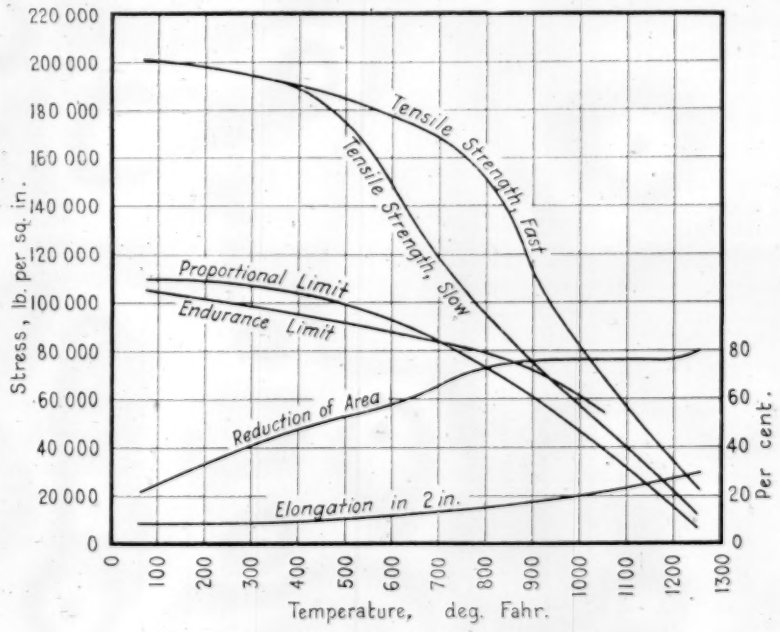


FIG. 2.—Experimental Results of Tests of a High-Carbon Steel Quenched and Tested at Various Temperatures.

At ordinary atmospheric temperatures steel is a crystallin substance which, within its elastic range under static load, acts as an isotropic, elastic substance. Under elevated temperatures, however, this general state of affairs no longer exists, and as the temperature is increased the metal gradually loses certain of its elastic properties and at the same time assumes a state approaching that of a plastic amorphous material. As this condition is approached, the steel tends to continually increase its stretch or strain without an accompanying increase of load, and the result is that the long-time tensile strength varies from the short-time tensile strength in an increasing percentage as the temperature is increased. The effect of this is well illustrated

in Fig. 2, which shows the variation of the static properties of a quenched metal and indicates the values of the tensile strength at elevated temperatures under ordinary test conditions and under long-time test conditions. In the short-time test the material was continuously loaded to its tensile strength within a period of about five minutes after it was raised to the correct temperature. The values of the tensile strengths in this case are shown by the upper tensile strength curve. In the long-time test, the specimen was tested to its proportional limit fairly rapidly and then increments of load were added only when strain or stretch had become zero, or almost so, for each increment of load. In this manner, the time necessary to

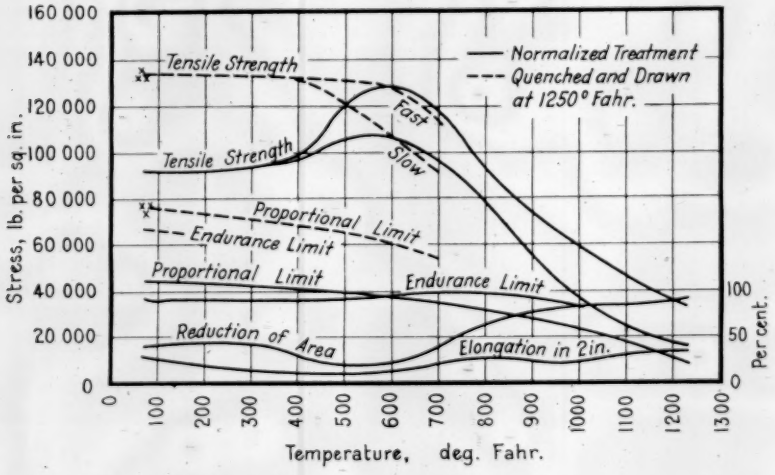


FIG. 3.—Curves Showing Static and Fatigue Properties of a 0.50-per-cent Carbon Steel Tested at Elevated Temperatures.

break a long-time test specimen varied from 12 to 72 hours, depending on the material and on the temperature at which it was being tested. The values of the long-time tensile strengths are shown by the lower portion of the tensile strength curve. It will be noticed that, as the values of the temperature of specimens are increased, the ductility values are also increased and the strength values are decreased. It should also be pointed out that the endurance limit curve goes above the long-time tensile strength curve as the temperature is increased. An explanation for this is given below.

A curious effect obtained in the testing of annealed or normalized steel is shown by the fact that, as the temperature approaches the neighborhood of the so-called blueing heat, the tensile strength is increased. Fig. 3 shows the static values obtained for this steel,

and it is suggested that this state of high stress is largely due to the temperature and stress conditions prevailing during the test, and is closely allied to the effect of mechanical working which it receives during the process of the test. The proportional limit is not increased as the temperature is increased, showing that the mechanical working has its greatest static effect during the yielding of the material at these blueing temperatures. In connection with this strengthening effect at these particular temperatures a few tests were made in which the metal was stressed to 90 per cent of the short-time tensile strength, then allowed to cool, and retested at ordinary temperatures. The results are indicated in Fig. 3 by the crosses at ordinary temperatures, and show that the effect of mechanical working is very appreciable and is retained when the metal is reduced to ordinary temperatures.

In order to demonstrate whether or not the strength of this steel could be obtained by heat treatment as well as by mechanical treatment at the blueing temperature, the best heat treatment, as found by previous tests in the Fatigue of Metals Laboratory,[1] was used for a series of static specimens. This metal was quenched and drawn at 400° F., as for treatment A A in the *Bulletin* referred to. The upper dotted lines in Fig. 3 show the static tensile strength of this metal at various temperatures and indicate that if a steel is heat-treated so as to develop its greatest strength at ordinary temperatures there is no hump in the strength curves at elevated temperatures, as is shown for the steel in the annealed or normalized state. It also shows that the tensile strength at the blueing temperatures of the heat-treated specimen is no greater than for the specimens that were normalized before testing. This shows that there is no advantage obtained in tensile strength by heat-treatment if the metal is to be used at or above the blueing temperature.

The dotted line in Fig. 3 next below the tensile strength line represents the proportional limit of the heat-treated steel at various temperatures, and is much higher than the proportional limit curve for the normalized steel. This bears out a previous statement to the effect that the strengthening effect on the normalized steel at the blueing temperature, due to mechanical working, comes after the yielding of the material, and is associated with the stress and temperature conditions during the approach of the test specimen to the tensile strength of the material. Retesting specimens at ordinary temperatures after they had been stressed to 90 per cent of the tensile strength and then cooled, shows that the proportional limit is raised

[1] H. F. Moore and T. M. Jasper, Second Report, Fatigue of Metals Investigation, *Bulletin No. 136*, University of Illinois Experiment Station (1923).

by mechanical working at the blueing temperature for all specimens after the initial stressing has been performed under such conditions. It is suggested that the reason for the foregoing phenomenon is that the blueing temperature of a wrought ferrous metal gives the best conditions for working a metal to obtain the maximum strength by mechanical means. That this strength can be made to approach that obtained by the best heat treatment, is indicated by those tests.

Fatigue strength results on steels at elevated temperatures, while essentially long-time tests, may be expected to exhibit phenomena somewhat different from that shown by long-time static tests. It has been shown that for temperatures above from 400 to 600° F. the static test results vary with the rate at which the specimens are stressed. The specimens used in obtaining the fatigue results at elevated temperatures shown in Figs. 2 and 3 have been stressed from maximum to minimum at the rate of 1500 cycles every minute, and this is probably comparable to the short-time static test rather than the long-time static test. It will be noticed in Figs. 2 and 3 that at high testing temperatures the fatigue endurance limits approach in one case the long-time static tensile strength, and in the other case exceed this value. It is expected that, as the rate of stress repetition is decreased in fatigue tests, the endurance limit found at elevated temperatures will also decrease, because a longer time will be available per cycle at the slower speeds for the yielding of the material, and the long and short-time effect will prevail in a manner similar to that shown in static tests.

The results presented in this paper are from tests which were performed partly at the University of Illinois and partly by the engineers of the Allis-Chalmers Manufacturing Co., and are also a part of the work undertaken by the Fatigue of Metals investigation. Acknowledgment should be given to the University of Illinois for the use of machines, to A. N. Talbot who is head of the Department of Theoretical and Applied Mechanics, to H. F. Moore who is in general charge of the materials and fatigue testing laboratories of the University, and to Mr. Harper and Mr. MacPherran of the Allis-Chalmers Co., whose results have been largely used for Fig. 1.

DISCUSSION ON PROPERTIES OF METALS AT ELEVATED TEMPERATURES

MR. J. KAYE WOOD[1] (*presented in written form*).—It is inspiring to know that in these days of feverish activity we have deep thinkers like Mr. Jasper, who, upon obtaining certain test results, consider their work only half done. **Mr. Wood.**

A comparatively great mass of data on the effect of elevated temperatures on the physical properties of metals has been accumulated in recent years due to the pressure of a generally recognized necessity. In this work some peculiar variations in the physical properties have been obtained consistently, which apparently has been the incentive for repeating the same investigations over and over again. Mr. Jasper appears to be the first person to consider seriously the "why" of these peculiar variations and if his paper does nothing else than to promote deeper thought along this channel it will deserve a prominent place in the history of this subject.

It is to be expected that the effect of time on the rate of increase in strain beyond the proportional limit would be more pronounced at elevated temperatures. There are many reasons to believe that this phenomena is due principally to the amorphous film surrounding the crystals. Steel is crystallin well beyond the temperatures considered in Mr. Jasper's paper, and even with metals like pure lead or tin, Ewen and Rosenhain have obtained brittle (inter-crystallin) fractures at temperatures ranging from 3 to 10° C. below the respective melting points of the metals.

Steel has a definite plastic phase as well as an elastic phase, even at ordinary atmospheric temperatures. Since the physical laws governing the plastic or amorphous phase are approximately the same as those governing liquids we might predict the general change in a metal due to increased temperatures and pressures.

Change Due to Elevated Temperature.—At ordinary atmospheric temperatures the viscosity of the plastic phase is very high, probably corresponding to a point on the straight portion of the viscosity-temperature curve, and at about 600° F. the viscosity probably begins to decrease at a fast rate. From this point on the decrease in tensile strength and increase in ductility would be as rapid as the decrease in

[1] Consulting Engineer, New York City.

Mr. Wood. viscosity of the amorphous film surrounding the crystals of the metal. As the viscosity approaches that of a true liquid the effect of time on the amount of deformation would be more pronounced.

Change of Pressure (relative).—In a paper on "Overstrain in Metals" presented before the Institute of Mining and Metallurgical Engineers, in February, 1924, the writer described a mechanism of combined elastic and plastic action in metals which accounts for the general shape of the stress-strain diagram. In this description it is shown that when a sufficiently high elastic force (stress) has been developed, the requisite pressure is thereby furnished to cause plastic flow. This stress corresponds to the proportional limit and since at room temperature the viscosity of the plastic phase is high, it is possible that the viscosity value is then quite well on the straight portion of the viscosity-pressure curve. This would account for the general indefiniteness of the proportional limit. On this basis it would require a lower stress at elevated temperatures to start plastic flow because of the lower viscosity. Thus a lower proportional limit at elevated temperatures.

Regarding Mr. Jasper's explanation of the increased tensile strength as the temperature approaches the neighborhood of the so-called blueing-heat, I believe it is a very plausible one. Between atmospheric temperature and 600° F. it appears that the viscosity of the amorphous film surrounding the crystals is sufficiently high to offer resistance to plastic flow within the crystals themselves. At about 600° F. and above very little resistance is offered, hence it would appear that just previous to this point it should be an ideal temperature range for performing cold work on steel. Considerably below this point severe distortion occurs, while above it there is no rigidity or cementing power in the amorphous layers to hold the most favorable orientation of the crystals. Specimens of iron and mild steel have been strained by Rosenhain at 900° F. without showing any great development of slip-bands within the crystals but there were clear signs of much movement in the crystal boundaries. In other words we lose tensile strength at elevated temperatures because, due to inadequate anchorage of the crystals, we fail to get the most work out of the crystals themselves.

Mr. French. MR. H. J. FRENCH[1] (*presented in written form*).—The paper by Mr. Jasper deserves special attention in that it relates to both endurance and static tests at various temperatures. No comments will be made concerning the data presented or the endurance limits obtained for carbon steels for the latter comprise a field of study which has so

[1] Division of Metallurgy, U. S. Bureau of Standards.

far received scant attention. However, exception may be taken to some phases of the discussion on static tests which may be misinterpreted; at least clarification of certain features would be helpful. **Mr. French**

It is unfortunate that reference is made, both in the synopsis and the text of the paper, to "long-time static tests" instead of adhering to the terminology used in the charts to differentiate between the customary tension tests and those in which the slow increases in load above the proportional limit required from 12 to 72 hours for fracture. Such slow loading experiments can hardly be classed with tests which Mr. Jasper states should "approach as nearly as possible the conditions under which the metal is to be used." In reality they cover very short periods compared to the life required in service and might more properly be referred to in some other way, such as, for example, "slow loading" tension tests.

The statement "that the blueing temperature of a wrought ferrous metal gives the best conditions for working a metal to obtain the maximum strength by mechanical means" is liable to be misleading. As pointed out by Jeffries and Archer,[1] the time required for an increase in the resistance to motion on slip planes during deformation "is less as the temperature rises so that at 250 to 300° C. ("blue heat"), the change is practically instantaneous." Whether "blue heat" gives the best conditions for working to obtain maximum strength by mechanical means is open to question.

According to Howe[2] "the loss of ductility as measured by endurance of bending and drifting is enormous" when structural steels are "blue worked," and earlier as well as more recent writers have repeatedly warned against the practice of working steels through the temperature ranges in which temper colors are obtained. While there are some differences of opinion regarding the deleterious effects produced, the author has not given sufficient evidence to justify the quoted statement.

If by best conditions he refers to the maximum possible tensile strength obtainable by working at various temperatures, a similar objection may be raised. Jeffries and Archer point out that "if the deformation which takes place during the tensile test at room temperatures could be effected very slowly so that there was time for elastic recovery during the test, it would be expected that the tensile strength would increase to an extent comparable with the increase obtained at a blue heat." In sustained loading tests recently completed by the writer and some of his associates, it was found that the

[1] Z. Jeffries and R. S. Archer, "The Science of Metals" (1924).
[2] H. M. Howe, "The Metallurgy of Steel" (1891).

Mr. French. "strain hardening" ability of low-carbon steel (and without doubt this applies to higher-carbon steels) is actually greater at atmospheric temperatures than at blue heat.

It should be made clear by Mr. Jasper that the rate of hardening at blue heat is the point at issue; blue heat cannot be considered as "the best conditions for working a ferrous metal to obtain the maximum strength by mechanical means" on the evidence presented in the paper.

In comparing "blue worked," normalized and quenched-and-tempered 0.50-per-cent carbon steel samples the following statement is made: "This shows that there is no advantage obtained in tensile strength by heat treatment if the metal is to be used at or above the blueing temperature." It is quite evident from Fig. 3 of the paper that the tensile strength values obtained on the heat-treated samples are approximately the same as those secured on the normalized samples at corresponding temperatures above "blue heat" but the quoted statement clearly implies that there will be no advantage in use to be gained from a preliminary heat treatment. This has not been demonstrated for no evidence is given in the paper to show that there is any relation between the load-carrying ability of the steel and the tensile strength at corresponding temperatures as determined in either the customary or "slow loading" tension tests. If, for example, comparisons for use under sustained loads are based on the proportional limits, Mr. Jasper's conclusion would be reversed throughout at least a portion of the temperature range above "blue heat." However, under long heating, the applied treatment would not be expected to give physical or constitutional permanence.

As will shortly be pointed out for hot-rolled low-carbon steel, carefully determined stress-strain diagrams and proportional limits are a better criterion of both long life and freedom from deformation under sustained loads than the tensile strengths such as used by Mr. Jasper.

In this connection it is significant that the endurance limits shown in Figs. 2 and 3 of the paper so closely approximate the proportional limits throughout a good portion of the temperature range covered. While somewhat higher at temperatures above 700° F. (370° C.) it has been pointed out in the paper that lower values are to be expected in the fatigue tests when the rate of stress repetition is decreased, that is, approaches sustained load conditions. In other words, sustained loads are more effective in reducing the life of the metal at the highest temperatures than the moderately rapid repetitions of the reported fatigue tests, for they give the greatest opportunity for "flow" or "creep."

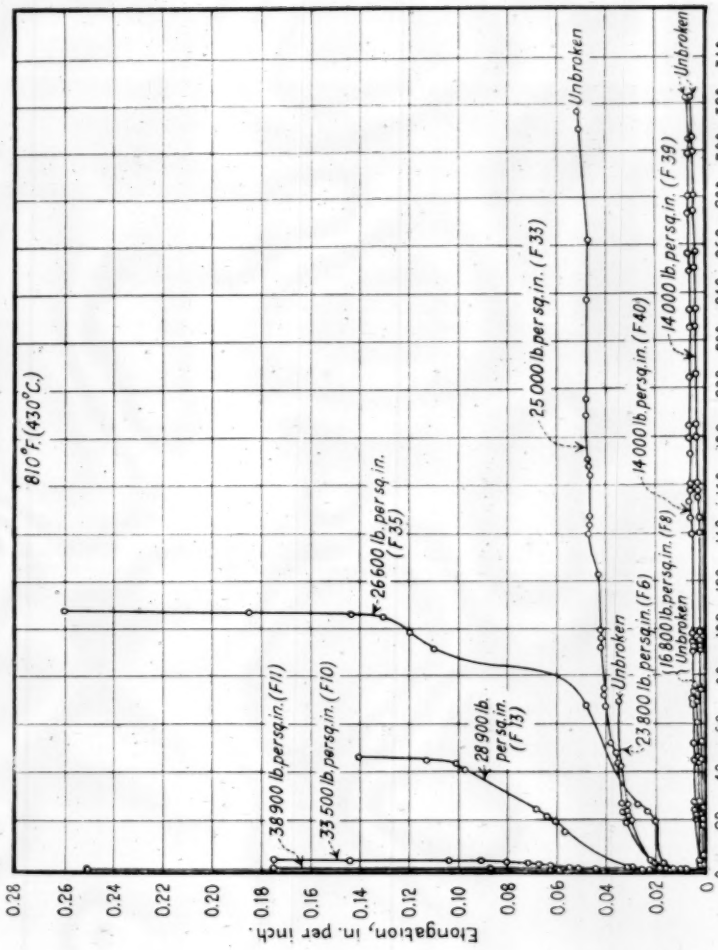


FIG. 1.—Flow in 0.24-per-cent Carbon Steel Under Sustained Tensile Loads at 810° F. (430° C.).

Similar and likewise more complete sets of curves have been obtained at other temperatures.

The numerical values placed on the curves represent the fixed applied loads in pounds per square inch of original cross-section.

Mr. French. During the past year the writer and some of his associates have studied the "creep" or "flow" of a low-carbon steel at various temperatures. As the results throw light upon some phases of Mr. Jaspers' discussion and will possibly be helpful to future investigators, a brief summary is here given; a more detailed report will shortly be published.

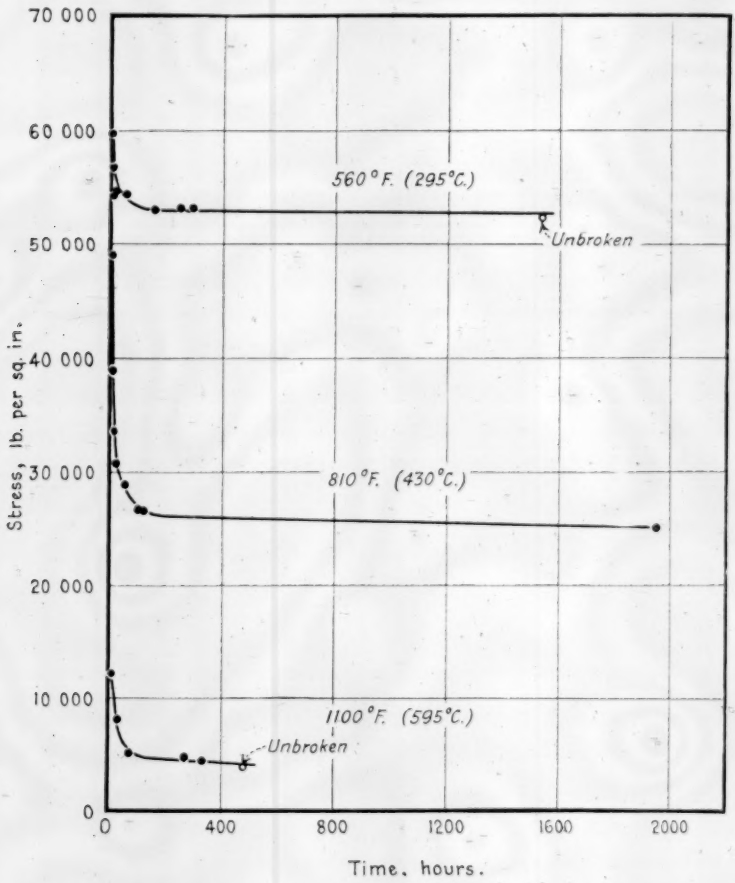


FIG. 2.—Relation Between Applied Load and the Life of 0.24-per-cent Carbon Steel at Various Temperatures.

In addition to the customary (short-time) tension tests, the flow was followed in 0.25-per-cent carbon steel when subjected to fixed applied loads at room temperature, 560° F. (blue heat), 810° F. and 1100° F. The time for which the loads were maintained varied with the conditions and the purpose of each test but extended up to 1800 hours.

Mr. French.

The total flow producing fracture when low-carbon steel is subjected to a fixed total load in tension at approximately constant temperature takes place in three distinct steps, the importance of which vary with the applied load and temperature (see the accompanying Fig. 1).

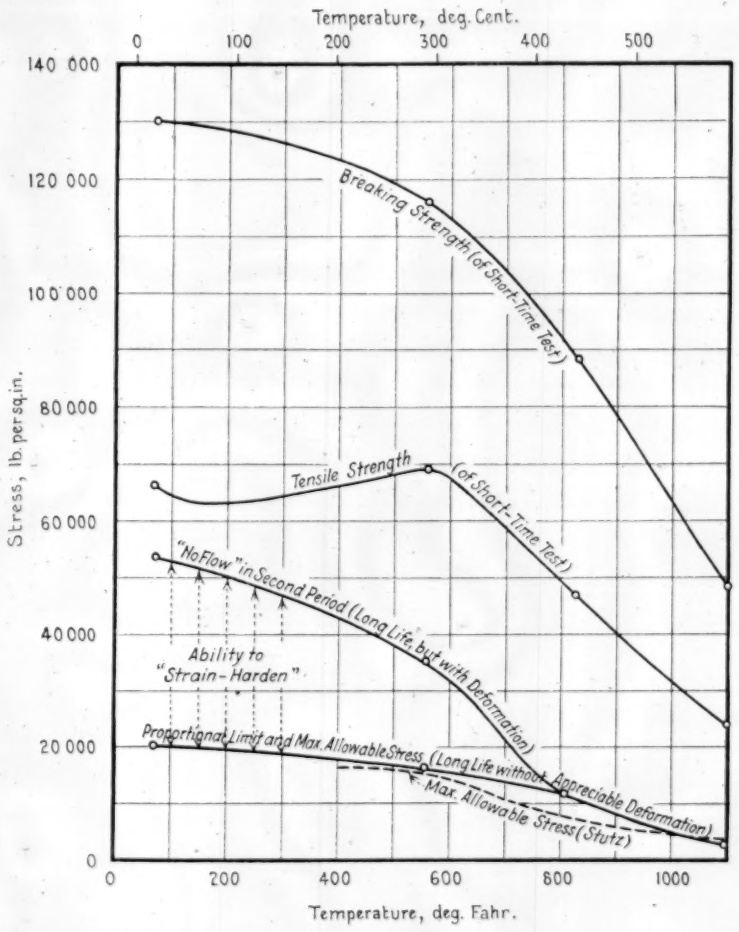


FIG. 3.—Comparison of Results Obtained with 0.24-per-cent Carbon Steel in Long-Time and the Customary Short-Time Tension Tests at Various Temperatures.

The three stages of flow are:

1. An initial flow.
2. A secondary flow at fairly constant rate which is also considerably less than the rate during the first and third periods.
3. A final rapid flow just before fracture.

Mr. French. As the constant applied load is increased, the initial flow and the rate of flow in the second period increase and the life of the steel decreases. The final rapid flow begins when the reduction in cross-section accompanying appreciable elongation has raised the unit stress to a definite value at each temperature.

The relation between decrease in applied load and increase in life is approximately hyperbolic (Fig. 2).

At atmospheric temperatures, there is a small difference in the loads permitting very long life and those producing fracture in a few moments. As the temperature is raised, the increase in life with decrease in applied load becomes more gradual. An important effect of temperature increase is to reduce the strain-hardening ability of the steel. This is a maximum at ordinary temperatures, and decreases until it becomes zero in the neighborhood of 750 to 800° F. (400 to 525° C.) (See Fig. 3). As a result, the principal factor governing the maximum allowable stress varies with temperature and the type of service. When both long life and freedom from appreciable deformation are required, the maximum allowable load closely approximates the proportional limit of the writer's "short-time" tests and similar tests at corresponding temperatures carried out independently at another laboratory. These proportional limits were found to closely approximate the maximum allowable stress values successfully used by an engineer for the design of commercial equipment operating at high temperatures. In the range 70 to about 600° F. (20 to 315° C.) higher working stresses can be used if appreciable deformation can be taken care of and long life is the primary requirement (see Fig. 3).

It is suggested that more attention be given by future investigators to the stress-strain relations when carrying out the customary short-time tension tests at high temperatures. Careful comparisons on the same material with other data such as the endurance test results reported by Mr. Jasper appear to offer more promise in developing for ferrous metals a relation between tests so readily carried out in the laboratory and what may be expected in various types of service than if emphasis is placed upon the tensile strength values as now determined.

Mr. Lynch. MR. T. D. LYNCH[1].—I should like to comment on this question of the properties of metals at high temperatures and the relation of the properties to the proportional limit. In 1908, a paper was presented by the speaker on the subject "The Use of the Extensometer

[1] Manager, Materials and Process Engineering Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

in Commercial Work,"[1] at which time this same thought was brought out and advocated with reference to steel at normal temperature, namely, that the proportional limit was the critical point about which designs must be made on materials that are worked at high stresses. **Mr. Lynch.**

At that time this conclusion was reached on account of some applications that were made of the pressing of armatures onto shafts at certain stresses, which seemed to be necessary in order to get the proper gripping effect, but it was found that the proper gripping effect was not realized and at that time it was determined that the proportional limit of the material used was a little below the pressing on stress, and for that reason the armature became loose on the shaft, and it was on account of that particular instance that the series of tests referred to were made, and from that time on we have continued the use of the extensometer in commercial work and the use of the proportional limit when applying material designed for high stress at normal temperatures. It is most interesting to the speaker to know that the proportional limit value of materials, a critical value at normal temperatures, is equally significant at elevated temperatures.

MR. T. MCL. JASPER (*author's closure by letter*).—The author is very gratified with the amount of discussion evoked by this paper and hopes that the matter of testing at elevated temperatures will receive increased support in order to develop test methods which may be reliable and not too time consuming. It is desired at this time to say that the typical test results presented here are on wrought ferrous metals and preclude all cast materials except one curve shown in Fig. 1 which is on cast iron. **Mr. Jasper.**

Mr. Wood has recognized a suggestion which I wish to emphasize, namely, that in the neighborhood of the blueing temperature, mechanical work will likely do most good from the standpoint of increasing the strength qualities of steel. It should be pointed out, however, that this mechanical work should be accompanied by a relatively small amount of mechanical strain and is not comparable to hot or cold working that is ordinarily used in reducing steel to various shapes.

Mr. French has in a sense looked at the matter from the viewpoint of the metallurgist and not so much from the viewpoint of using the wrought metal for constructing machines and structures to be used at elevated temperatures. I would therefore take issue with him in his statement that, "It should be made clear by Mr. Jasper that the rate of hardening at blue heat is the point at issue." From the

[1] T. D. Lynch, "The Use of the Extensometer in Commercial Work," *Proceedings*, Am. Soc. Testing Mats., Vol. VIII, p. 640 (1908).

Mr. Jasper. metallurgist's point of view and from the exceedingly complicated and intense discussions on theories which have grown up between metallurgists this may be important, but from the viewpoint of the engineer and of the user of steel at elevated temperatures, the physical properties of metals at temperatures at which they are to be used together with the development of reasonably accurate methods of testing at such temperatures and the consideration of the consumption of time for making such tests are the points at issue and should not be lost sight of.

Mr. French in general has suggested that the method of making the tests reported in this paper do not approach as nearly as possible the conditions at which the metals are to be used. This is probably true so far as the element of time is considered, but I wish to point out from his Fig. 2 that stressing from 50 to 100 hours gives the ultimate strength within a small percentage of what is found if the specimens are stressed for a period in the neighborhood of 2000 hours at various elevated temperatures. For over three years the author has been running long-time tests at ordinary temperatures on various metals in which over 2000 hours were consumed by each specimen in raising the stress from 80 per cent of the short-time ultimate strength to the eventual ultimate obtained. These tests indicate for ductile low-carbon wrought ferrous metals that the test consuming but a few minutes gives values which are within 3 per cent of the long-time ultimate strength. As the temperature of testing is elevated, however, the short-time values vary in a larger and larger percentage from the long-time values obtained by the method adopted by the author. The author by adopting a method of adding load only after strain had stopped or nearly stopped intended to approach within a small percentage of the possible ultimate strength when a large number of hours were consumed in applying each increment of load. Whether this has been accomplished remains to be demonstrated.

The author would also like to emphasize the possible fact that little or no advantage is gained by heat treating a steel that is to be used at or above the blueing temperature. It should be clearly understood that so far as the experiments reported here are concerned wrought ferrous metals only are considered. In Fig. 3 of the original paper is shown a dotted line representing the proportional limit of the quenched-and-drawn steel tested at various temperatures. This line is above the continuous line representing the proportional limit of the normalized steel at various temperatures. In this series of tests two specimens of the normalized steel were used at 600° F.

which were not reported in the original paper and which were tested to 90 per cent of the short-time ultimate strength, the load released, and the test immediately run over to destruction. The average proportional limit obtained on the two specimens referred to above after the preliminary pull had been made gave a value slightly below 60,000 lb. per sq. in., which is a value similar to that shown by the heat-treated steel in this series of tests. In considering the typical tests reported in this paper it should be said that Figs. 2 and 3 of the paper are selected from experiments covering several sets of steel of various carbon content and heat treatment and two non-ferrous metals, the desire being in presenting this paper to eliminate confusion which would occur by reporting several sets of tests and to confine the discussion to wrought ferrous metals. **Mr. Jasper.**

The author wishes to state frankly that he does not understand Fig. 3 of the discussion by Mr. French. In the tensile strength curve shown in this figure it is presumed that the plotted points represent actual values. If this is the case it would seem that the interpolation used is problematical. It would be helpful if the curve immediately below the tensile strength curve were explained more fully.

MR. V. T. MALCOLM[1] (*by letter*).—The effect of temperature on the properties of metals is now recognized to be of great importance in the design of equipment, especially for the modern steam power plant and oil refinery. This is of importance to the designer, the manufacturer and the user of such equipment, since they should all be familiar with the physical characteristics of metals or alloys at temperatures to which the metals or alloys will be subjected in service. **Mr. Malcolm.**

A considerable amount of information regarding the properties of metals at elevated temperatures was presented in 1924,[2] but most of the data was obtained by the short-time test. At that time the writer stated that a knowledge of the behavior of metals at elevated temperatures over periods of long duration would undoubtedly be of great technical importance for the reason that long-time tests, as reported by Dickenson,[3] had tended to create grave doubt as to the validity of our ideas on yield point and elastic limit of certain metals in common use. Since design is generally based on yield point (the so-called commercial elastic limit), the matter becomes important. We have noted in several cases of metals that had failed that a gradual change of structure occurs after a lapse of time, and this is one reason for failure of metals that had shown excellent results when tested in

[1] Metallurgist, Chapman Valve Manufacturing Co., Indian Orchard, Mass.

[2] "Symposium on Effect of Temperature Upon the Properties of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 9 (1924).

[3] *Engineering* (London), Vol. 114, pp. 326–378, September, 1922.

Mr. Malcolm. a short-time test at elevated temperatures. Tests of short duration are not always sufficient to indicate the behavior of a metal in service, and it should be our aim to carry out such tests as will approximate the condition of service. We are, therefore, greatly indebted to the authors and to those who entered into the discussion, for the valuable information contributed.

Because of the importance attached to the securing of a better knowledge of the behavior of metals at elevated temperature over periods of long duration, the research laboratory of our company

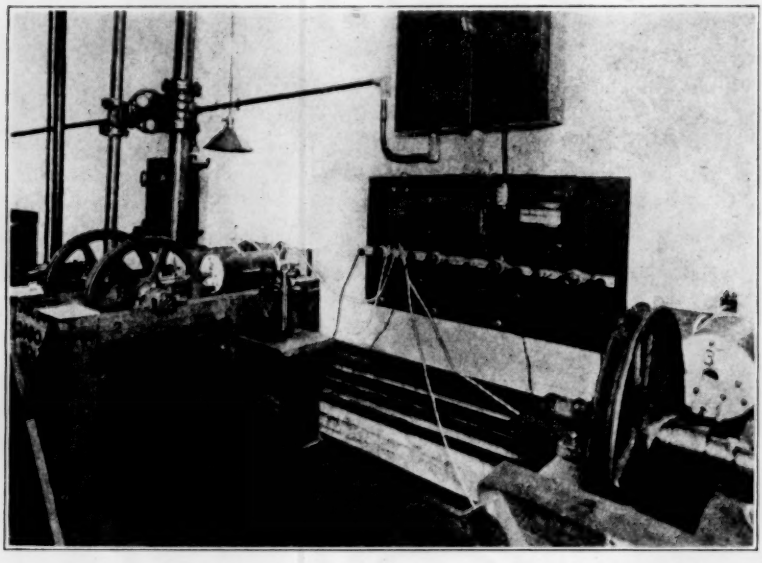


FIG. 4.—Tension-Test Machine for Long-Time Test.

began a systematic study of the properties of metals at elevated temperatures. I believe that we are the first to carry out this line of research on cast steels and we hope from the results obtained to show the possibilities of cast steel in the design of structures operating at elevated temperatures.

Considerable work has been completed and published during the past several years on the short-time testing of metals at elevated temperatures, and late in 1924 long-duration tests were started. Much of the work has been completed, especially on cast carbon and cast chromium-nickel steel.

Apparatus.—The apparatus used in carrying out these tests is shown in the accompanying Figs. 4 and 5, and consists of electrically

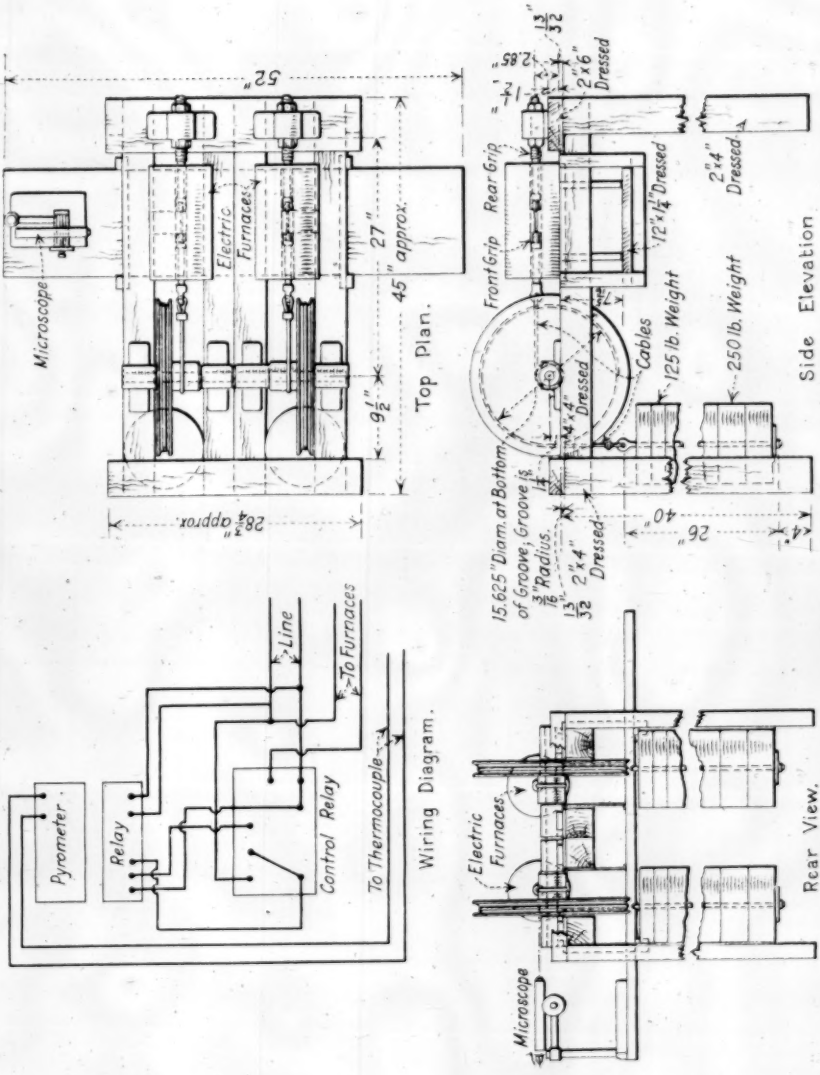


FIG. 5.—Tension-Test Machine for Long-Time Test.

Mr. Malcolm. heated furnaces, in which the test bars are placed, which bars are screwed into adaptors attached to cables, the opposite ends of these cables passing over a drum. An 18-in. wheel carries another cable, with load suspended, making the entire system in a horizontal plane. The temperature control is obtained by an automatic controller, which keeps the temperature of furnace constant during the duration of the test.

The sides of the furnaces are slotted and fitted with doors, through which the "creep" of the test bar is measured with a micrometer

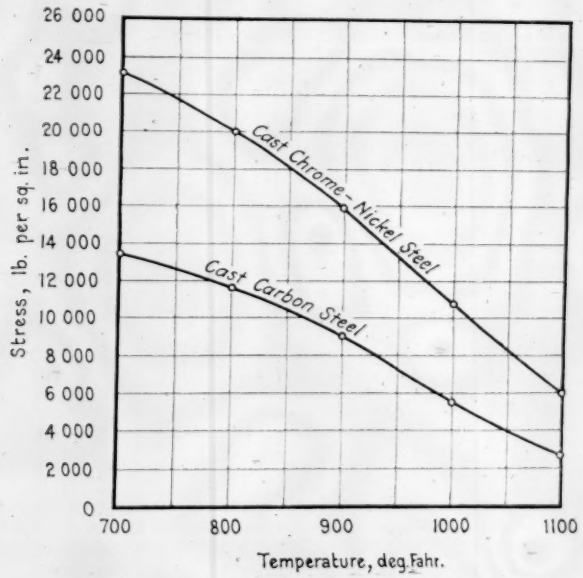


FIG. 6.—Maximum Allowable Stress, Long-Time Tests, Long Life Without Deformation.

microscope graduated to 0.0001 in., so that very small movements in the steel under test may be determined. Four units are in operation at the present time, and four more are in course of construction.

Discussion of Results.—The results reported by Messrs. Lynch, Mochel and McVetty and by Mr. French, have been confirmed by the writer. The writer believes that at each temperature there is a particular stress at which the metal may flow and that it continues to deform under continuously applied stress, but with time the "creep" will become constant. Above this stress, the metal will "creep" rapidly until final fracture occurs.

Our results show that for each temperature of steel tested, there is at this temperature a stress which if exceeded will cause continuous

"creep" and finally fracture. Though valves may operate under this stress and this temperature for long periods, failure will finally result. It was therefore necessary to determine the stress at each temperature that can be applied to cast steels without causing appreciable "creep" **Mr. Malcolm.**

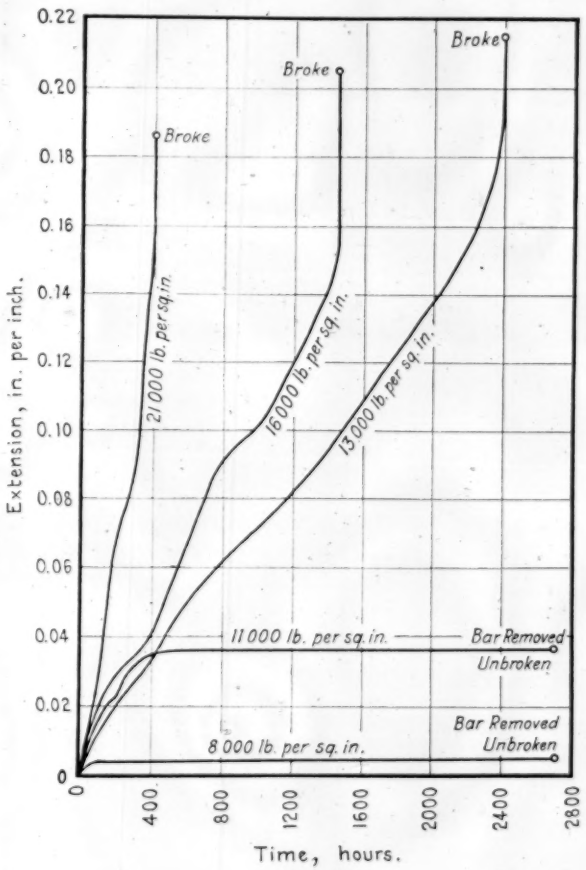


FIG. 7.—Long-Time Tests on Cast Chrome-Nickel Steel, Showing Life Under Sustained Loads at 1000° F. Constant Temperature.

or deformation. From the results obtained in our investigations we feel perfectly assured that cast alloy steel will stand the severe service of 1000° F. without deformation over long periods, provided the allowable stresses shown in Figs. 6 and 7 are not exceeded.

These tests have been corroborated by practical tests in the field on full sized valves and fittings.

Mr. Malcolm. The only caution the writer advances at this time is that anyone contemplating tests of this type should have complete knowledge of his metals and testing machines, before he begins to apply the results of his tests to practical work.

In concluding, the writer wishes to emphasize the importance of the tests under discussion, and believes that in the near future, from results of these tests, a relation will be established between the long and short-time tests (this relation has already been fairly well determined) that will provide the engineer with a rapid method of determining the strength of his metals at the temperature to which they will be subjected, and the result will be that the best materials for specific application will be obtained.

Messrs. Lynch, Mochel and McVetty. MESSRS. T. D. LYNCH, N. L. MOCHEL AND P. J. MCVETTY (*author's closure by letter*).—The contribution by Mr. French was offered mainly in discussion of Mr. Jasper's paper, but actually applies to our own as well. He is to be congratulated on the splendid work, and the data presented. His Fig. 3 is of especial interest and value. The statement that "when both long life and freedom from appreciable deformation are required, the maximum allowable load closely approximates the proportional limit of the 'short-time' tests" confirms the opinions of other investigators that a study of the stress-strain relationship yields valuable information and that the proportional limit as determined by short-time tests will probably be found reliable in estimating the ability of any ferrous material to bear sustained loads. It is observed in Fig. 3 that the "infinite life" curve (long life but with deformation) joins with the proportional limit curve at 810° F. and coincides with it from 810 to 1100° F. We would suggest that this is true of the 0.25-per-cent-carbon steel examined; it may not be true for other steels, the merger may be above or below 810° F.; and there appears ample evidence that in the case of certain non-ferrous materials which we have examined, that this curve of "infinite life" may actually cross the proportional limit curve and continue lower than that curve.

We wish to thank Mr. Malcolm for his discussion and for the data presented. His "long-time" test apparatus shown in Fig. 4 deserves much commendation, as it shows considerable ingenuity. The optical measuring device should give very splendid results, and was suggested by one of the authors several years ago, but was not used in our own work as the Martens extensometers which had been developed were more delicate and gave finer results. It is observed that measurements are taken on one side of the specimen only, and it would appear to us that more accurate results would be secured if measurements were taken on two sides of the specimens. His wiring diagram for furnace

temperature control is shown, and we would point out that this is a matter of great importance and one deserving the utmost care on the part of all investigators. One cannot make the control apparatus too exacting. **Messrs. Lynch, Mochel and McVetty.**

Referring briefly now to Mr. Jasper's paper, he has emphasized in both paper and closure that little or no advantage is gained by heat treating a steel that is to be used at or above the blueing temperature. He does not, in our opinion, present data in support of such a claim. His specimens were quenched and drawn back to but 400° F., and tests were apparently made up to 700° F. One would naturally expect a tempering effect immediately upon the specimen being heated above 400° F., and reference to the tensile strength curve in his Fig. 3 clearly shows the falling off in his so-called "slow" test to start rapidly at 400° F. We suggest the variables, introduced by testing at temperatures higher than the drawing temperature, are sufficient to discredit the above claim. We would change the reading of his claim that there would be little or no advantage in heat treating a steel that is to be used at or above the drawing temperature given the steel.

The tempering effect also enters in and prevents acceptance of the statement that if a steel is heat treated so as to develop its greatest strength at ordinary temperatures, there is no hump in the strength curves at elevated temperatures as is shown for the steel in the annealed or normalized state. The impression is left that in the heat-treated state, a hump is never experienced in the strength curves. Our Figs. 13 and 15 clearly show such a hump for water-quenched-and-drawn carbon steel and oil-quenched-and-drawn 5-per-cent-nickel steel.

Regarding the endurance values given by Mr. Jasper, it is to be regretted that space was not taken to show the apparatus and methods used in carrying out these tests. He has entered on a field of work that has received but little attention, and complete information relative to apparatus and methods would have been of great interest and much value.

We are inclined to agree with Mr. French in suggesting that Mr. Jasper would have done well to call his "long-time" tests by some other name as they do not fall in line with the now more or less accepted meaning of the term "long-time" tests. Further, in making his tests, he states that after passing the proportional limit, increments of load were added only when strain had become zero, or nearly so, for each increment of load, and yet the tests were completed in from 12 to 72 hours. Information is not given as to the type or accuracy of the extensometer used, but if sensitive measuring apparatus had been employed and increments of additional load not applied until stretch

Messrs. Lynch, Mochel and McVetty.

had become zero, it is inconceivable that the tests could have been made in 12 or 72 hours. Our Fig. 19 shows that loaded directly at the proportional limit, stretching was continuing after 1200 hours with no sign of any let-up. Mr. Malcolm confirms this condition. The writers also agree with Mr. French that the "long-time" method employed by Mr. Jasper does not appear to approximate conditions of service.

Mr. Jasper.

MR. T. MCL. JASPER (*author's closure to discussion by letter*).—The writer would like to comment briefly upon the point raised by Messrs. Lynch, Mochel and McVetty, who have questioned a statement in his earlier closure. The statement is as follows: "The author would like to emphasize the possible fact that little or no advantage would be gained by heat treating a steel that is to be used at or above the blueing temperature." The writer has had to recognize the fact that above the blueing temperature changes go on in steel under stress which seem to be akin to that of annealing. This suggested that annealing can be performed below the first critical point in steel if a sufficiently long time is taken. This leads to an observation which recognizes the possibility that in the long-time test, strain values at high temperatures progressively increase due to two factors: one a normal creep due to load which may be of short duration and the other a creep due to the change in the character, and consequently a change in strength properties, of the metal which parallels the rate of low temperature annealing at the particular temperature used. The rate of change of such strength properties may vary when considering alloy as against carbon steel.

This leads to a suggestion for a series of experiments which would have for its aim the long-time testing, at elevated temperatures, of steels that had been thoroughly annealed and comparing with similar tests of the same steels which had been heat treated to see if at high temperatures the ultimate strengths coincided or nearly so.

At the blueing temperature, the writer is satisfied as to the effect of temperature and stress on normalized or annealed carbon steels, that is, their strength increases and coincides very closely with that of the best heat treatment of the steel. If initial straining is performed at this temperature to within a small percentage of the ultimate strength of the metal and the specimens allowed to cool off and then are subsequently tested at various temperatures below the blueing temperature, the results parallel those obtained from specimens which were primarily given the best heat treatment and tested at various temperatures.

Mr. Jasper.

What would similar tests on specimens of steel which have been held at a temperature much above the blueing temperature for 1200 to 2000 hours while under stress show? Will the second test give the same elastic limit as did the original test at the same temperature? The writer is of the opinion that it would not and is also of the opinion that the value would be lower in the second case due to the long-time annealing effect. He knows that the material tested in this manner at temperatures below the blueing temperatures has its elastic limit raised in the second test to a value above that of the first test provided it is first prepared as outlined in the preceding paragraph.

The writer would like to point out from the paper by Lynch, Mochel and McVetty that there are presented no short-time or semi-long-time test results to compare with the exceedingly long-time data to see how far apart the various methods of testing really are. It seems that this is necessary before a test method can be correctly evaluated. The writer does not wish to depreciate the value of the very long-time test but is satisfied that it is really a means to an end. The end is to set reasonable limits of time in testing steels at elevated temperatures. To use a very large amount of time on elevated temperature tests when the shorter methods might give results for all working temperatures within a relatively small percentage of the long-time values, is a waste of valuable time and is a retarding factor in the obtaining of valuable information on many more metals and heat treatments which could not be done if single tests ran from 1200 to 2000 hours.

A statement in the writer's original paper is questioned, namely, the statement "that if a steel is heat treated so as to develop its greatest strength at ordinary temperatures there is no hump in the strength curves at elevated temperatures as is shown for the steel in the annealed or normalized state." This is not merely a statement but an observation obtained from the test results presented in the original paper. The data is obtained by quenching a carbon steel in water from above the critical point and by testing at no draw and at successive draws of 400, 600, 800, 1000, and 1200° F., and from a curve obtained in this manner, selecting the best heat treatment for strength, and testing the steel so selected at various temperatures up to the blueing temperature. This kind of experiment has now been performed for three different kinds of carbon steel, with the exception that the higher draws (above the blueing temperature) have not been carried out, and the writer has no reason to modify the original statement. The writer would suggest such a series of tests by those questioning the statement.

With reference to the endurance limits obtained at elevated

Mr. Jasper. temperatures, these tests, together with the apparatus used, are described in the latest Bulletin on Fatigue of Metals under the joint authorship of H. F. Moore and the writer.[1]

It is not the purpose of the writer to offer a closure that has any attitude in it but that of a helpful cooperative discussion—a presentation of ideas supported by experimental fact and correlative observations drawn from such facts. He is exceedingly gratified by the amount and quality of the discussion evoked.

[1] *Bulletin No. 152*, Engineering Experiment Station, University of Illinois.

FATIGUE OF METALS BY DIRECT STRESS

By Paul L. Irwin[1]

Synopsis

Fatigue tests on specimens which have been subjected to a bending action furnish endurance limits which constitute valuable information to the designer. Considerable data on this subject are available and working stresses can accordingly be based upon these results. It is essential, however, to know something about the endurance limit of metals under direct stress (cycles of tension-compression) and, what is more important, to have some data on the endurance limit for cycles of stress other than those of complete reversal. The direct stress method of testing lends itself to this work and is therefore valuable to the engineer.

This paper discusses a new apparatus, which has been designed as an improvement to direct stress fatigue testing machines, together with the endurance limits obtained as compared with those resulting from flexural stresses.

Introduction

The elastic limit in tension has been the criterion governing the dimensions of machine parts regardless of the nature of the stresses imposed, that is, whether they be due to live or dead loads. For a structure under a dead load or one subjected to stresses which are varied or reversed but a few times during the life of the structure, the elastic limit in tension is the determining factor in design. For machine parts which must endure millions of reversals of stress, however, the endurance limit is the important physical property. It is a limiting stress below which a metal may be stressed indefinitely without fracture.

Up to the present time the fatigue properties of metals have been obtained principally by the bending type of test. The good feature of this test is that it is easy to make. There are certain limitations in this test, however, which make tests under other types of stress, such as cycles of tension-compression, imperative. The limitations of the bending type of test are:

1. We are held to a stress cycle of equal plus and minus values, the top of the specimen being in tension and the bottom in compression to the same extent. Various ratios of these stresses occur in practice and a study of them is important.

[1] Research Department, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

2. If the proportional limit of the material is below the endurance limit, the beam formulas for stress do not hold true since we no longer have a straight line relation between stress and strain. If endurance limits are based upon such tests they are open to criticism.

3. Assuming that the proportional limit of the material is above the endurance limit and that the straight line relation between stress and strain does hold, we are overstressing the outer fibers of the test piece while the inner part may become strengthened by stressing it just below the endurance limit.

These doubtful points associated with the bending type of test are not present in the direct stress tests in which the whole area is subjected to an accurately known stress. The difficulty encountered in this test has been in obtaining a purely axial load. The author has succeeded in designing a new type of alignment seat and special test piece grips which have overcome this difficulty. Comparative tests on the direct and flexural types have shown that there is good agreement between the two methods. Therefore, we are justified in using the easier bending test to obtain endurance limits if the proportional limit is above the endurance limit and where the endurance limit under complete stress reversal is sufficient.

DESCRIPTION OF MACHINE

The machine used for obtaining the endurance limits in direct stress was that designed by B. P. Haigh and made by Bruntons. In this machine the test specimen is subjected to a tensile and compressive stress alternating at 2000 cycles per minute. The upper end of the test specimen is held stationary while the lower end is threaded into a cross-head. This cross-head is constrained to move in an axial direction relative to the test piece. The motive power is supplied by an armature which is attached to the cross-head and which moves through a small air gap between two fixed magnets. The machine was supplied with threaded bushings in the head and cross-head into which the test specimen could be screwed. It is evident that the center line of the head, the axis of the specimen and the line of motion of the cross-head must all lie in the same straight line if the test specimen is to be subjected to an axial load. Preliminary tests with a Marten's extensometer showed that due to the grips being non-adjustable, considerable bending stresses could occur in the test specimen. Alignment seats and test-piece grips were designed to correct this condition. Before these are discussed it is necessary to say something about the type of test specimen employed.

TEST SPECIMEN FOR DIRECT STRESS

In this type of machine the total range of load is 2700 lb. A specimen subjected to equal tensile and compressive stresses is impressed with a load of half this value, or 1350 lb. For small loads, certain discrepancies occur in the magnetic characteristics and it is necessary to work on the upper two-thirds of the load scale in order to obtain reliable results. This makes it necessary to have different sizes of test specimens in order to test materials varying widely in endurance limit values.

The diameter at the minimum section was varied for different materials as follows:

MATERIAL	MINIMUM DIAMETER, IN.	AREA, SQ. IN.
Low-carbon steel	0.226	0.040
Medium-carbon steel	0.196	0.030
Chrome-nickel steel	0.160	0.020
Manganese bronze	0.253	0.050
Cast iron	0.253	0.050

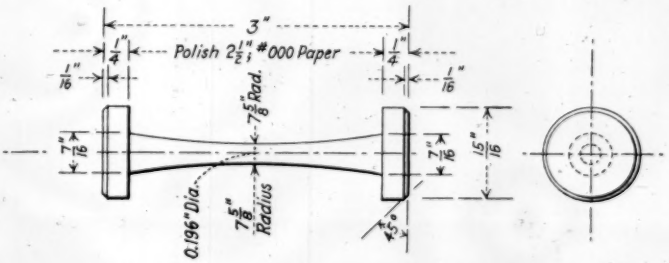


FIG. 1.—New Form of Test Specimen.

It was furthermore thought desirable to change the design of the test specimen in order to facilitate assembly, avoiding any stress concentration on the central portion and still retaining a certain amount of rigidity in order to withstand the compressive loads. In order to fulfill these requirements the original type of specimen, with threaded ends and reduced section, was modified to that shown in Fig. 1. The test specimen between the heads is formed by swinging the cutting tool on a 7⅝-in. radius. Fracture always occurs in the central section of the specimen, although the stress is not concentrated.

NEW ALIGNMENT SEATS AND GRIPS

In the new arrangement, ball-and-socket alignment seats were embodied to allow freedom of the test piece from bending stresses.

Having assumed an unstrained position, the balls are locked against the seats by a half turn of a nut.

On account of the difficulty encountered in the insertion of a threaded test specimen and the consequent probability of overstrain of same, the button-head type of test piece was designed in conjunction with the grips which are screwed permanently into the ball joints. A few turns of a nut lock the button heads in the grips.

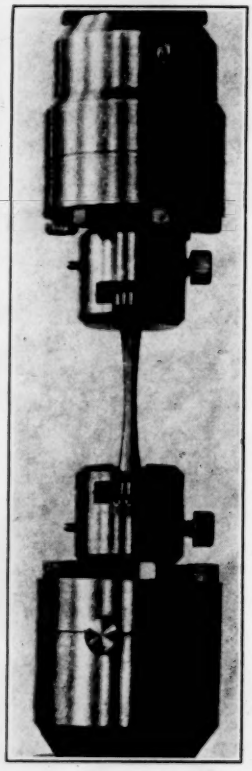


FIG. 2.—Alignment Seats and Grips.

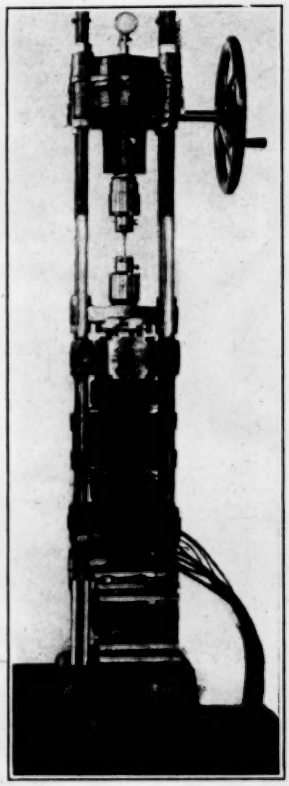


FIG. 3.—Ball Seats and Grips Installed.

The grips are shown in Figs. 2 and 3. These grips lock the test specimen against vibration, give a self-centering fastening to ensure concentricity between ball joint and axis of test specimen, give rapidity in insertion of test specimen and prevent lateral deflection of the specimen. The use of such grips has proved satisfactory and a fractured test specimen may be removed and a new specimen assembled in two or three minutes as against about thirty with the old arrangement.

The degree of perfection of the alignment obtained may be estimated by examination of the fractured test specimen. If, for example, a low-carbon steel be subjected to a stress which will cause fracture to occur at approximately 100,000 cycles, there will, at failure,[1] be a noticeable extension of the specimen in the region of minimum cross-section. The cracked test specimen on being rolled upon a surface plate is observed to roll true, thus showing that, through the use of the new grips, no bending stresses have been acting—otherwise

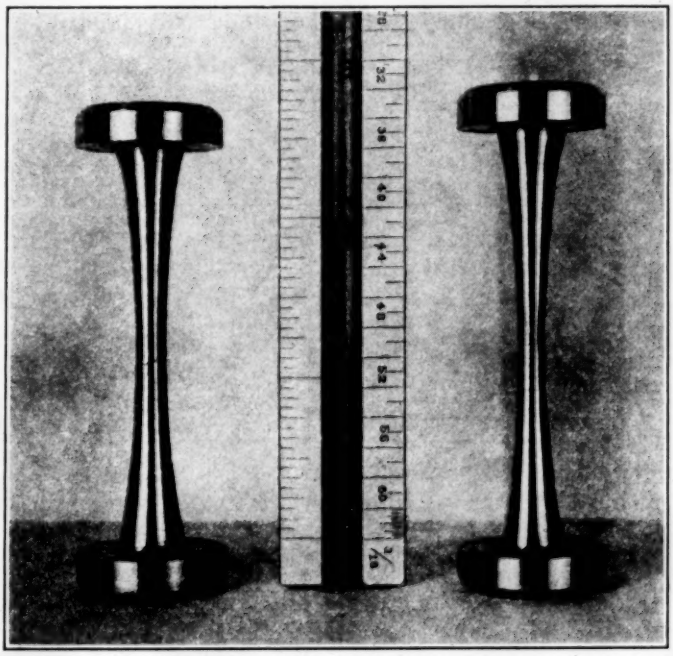


FIG. 4.—Fractured Test Specimens.

the cracked specimen would have become bent. Two fractured test specimens are shown in Fig. 4.

A further proof of alignment may be seen from the appearance of the fatigue test curves themselves. For the cases examined (low-carbon, medium-carbon and chrome-nickel steels) the curves obtained by plotting stress against number of cycles to fracture were very regular, the points all lying on a smooth curve. To such a degree was this regularity possible that having obtained some points on the curve for high stress values and some for low stress values then, on

[1] A circuit breaker controlled by a 6-volt relay actuated by a switch on the moving armature could be so adjusted that the machine was stopped when partial fracture had occurred.

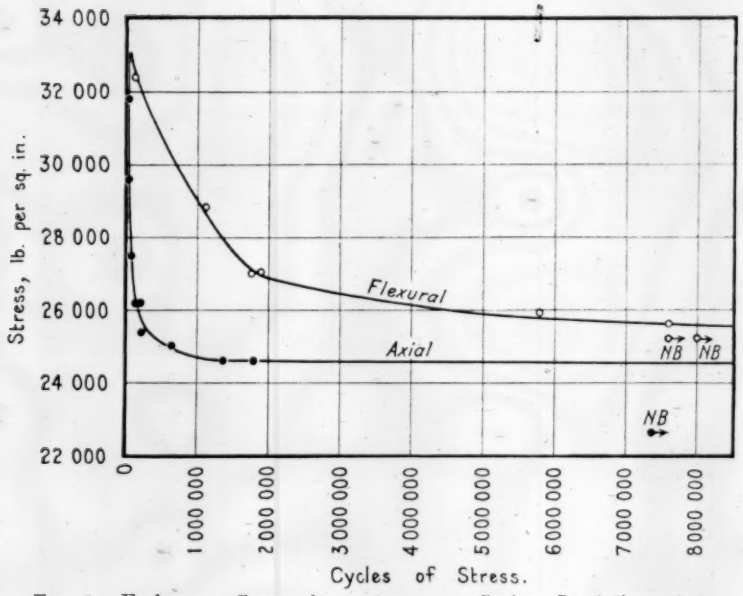


FIG. 5.—Endurance Curves for 0.15-per-cent Carbon Steel, Annealed.

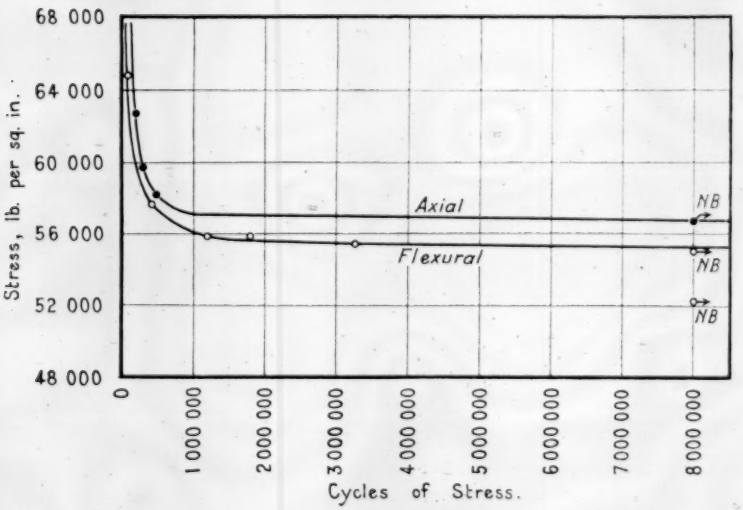


FIG. 6.—Endurance Curves for 0.68-per-cent Chromium, 2.93-per-cent Nickel Steel.

running test specimens at intermediate stress values, fracture always occurred at that number of cycles shown by the curve. This was never found possible with the bending type of test specimen and shows that alignment had been obtained. Such curves are shown in Figs. 5 to 7.

CALIBRATION

The next development in this apparatus was with regard to the calibration test bar. Due to the load being applied magnetically, calibration was deemed necessary. The original calibration test bar

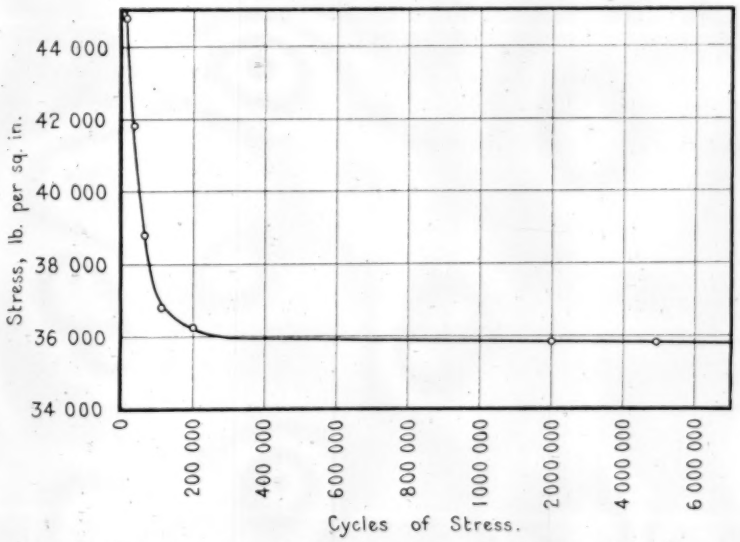
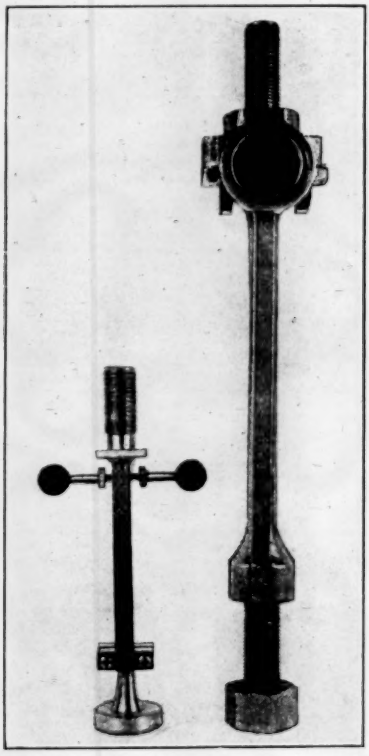


FIG. 7.—Endurance Curve for Medium-Carbon Steel Under Axial Stress.

supplied with the machine is the larger of the two shown in Fig. 8. The image of a lamp filament is reflected by the mirror upon a ground-glass scale attached to the rear of a camera box. The strain was measured over a length of the test bar which included the parallel middle part in addition to fillets and was magnified optically by the moving mirror and camera, the maximum length of the band of light being 3 in.

In the preliminary tests it was found that the moving part of the mirror arrangement of the calibration bar as discussed above was too heavy to enable strains to be recorded accurately at the normal frequency of 2000 cycles per minute. The smaller calibration test bar shown in Fig. 8 was accordingly designed by the author. The

principle of operation was taken from that of the Martens mirror extensometer. Two 90-deg. V-grooves, 0.010 in. deep, give a definite gage length of $2\frac{1}{2}$ in. A loop of No. 18 copper wire twisted into the upper groove definitely locates the fixed end of the comparison strip. A fine V-groove on the under side of the comparison strip, at a distance of $2\frac{1}{2}$ in. from the wire, serves as a seat for the outer knife



New Calibration Test Bar. Haigh Calibration Test Bar.

FIG. 8.

edge to the axis of which the mirrors are attached. The lower V-groove in the test bar holds the opposite knife edge in place. The ends of the knife edge rhomb are threaded to take the tapped mirror carriers. A small nut is used to lock the mirror carriers in position. It will be observed that the upper end of the calibration bar has a button head while the lower end is threaded and supplied with a square section to take a wrench. By eliminating the lower test specimen grip and screwing the calibration bar directly into the lower ball,

sufficient overall length is obtained to permit the use of a 2½-in. gage length without disturbing the setting of the upper head of the machine, and retaining the alignment properties of the ball joints.

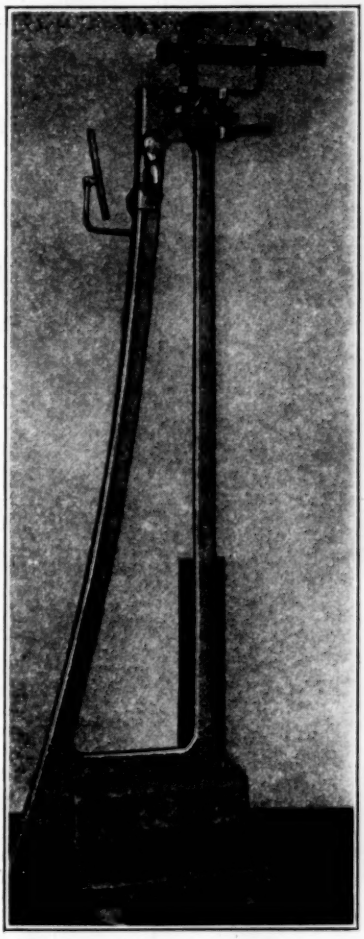


FIG. 9.—Strain Measuring Apparatus.

The deflection apparatus shown in Fig. 9 is used to measure the amplitude of the angle through which the mirrors are tilted due to the strain in the test specimen, and is placed about 100 in. from the mirrors. Light from the galvanometer lamp (which may be seen mounted on a slider at one side of the curved scale) is reflected by the

mirror through the fixed telescope. The band of light is clear and well defined and has a maximum length of over 10 in. at 100-in. radius. The inertia of the moving parts appears to be negligible.

RESULTS OF TESTS

In certain tests which have been made using the alignment seats and test piece grips described, a variety of materials were used, the tensile properties of which are given in Table I. It will be noted that these properties vary over a fairly wide range.

TABLE I.

Material	Proportional Limit, lb. per sq. in.	Yield Point, lb. per sq. in.	Tensile Strength, lb. per sq. in.	Elongation in 2 in., per cent	Reduction of Area, per cent
0.15-per-cent Carbon Steel (Annealed)	30 870	32 000	49 400	41.0	69.1
0.37-per-cent Carbon Steel (Annealed)	39 000	42 000	80 500	28.1	42.0
0.68-per-cent Chromium, 2.93-per-cent Nickel Steel	100 000	113 000	133 700	18.4	49.7
0.84-per-cent Chromium, 3.33-per-cent Nickel Steel	92 000	97 000	119 000	24.5	59.4
Manganese Bronze	32 500	79 500	105 500	8.0	17.0

TABLE II.

Material	Endurance Limits		Ratios	
	Direct Stress, lb. per sq. in.	Flexural Stress, lb. per sq. in.	Direct / Flexural	Direct / Tensile Strength
0.15-per-cent Carbon Steel (Annealed)	24 500	25 500	0.96	0.50
0.37-per-cent Carbon Steel (Annealed)	33 000	30 000	1.10	0.41
0.68-per-cent Chromium, 2.93-per-cent Nickel Steel	56 200	55 000	1.02	0.42
0.84-per-cent Chromium, 3.33-per-cent Nickel Steel	58 200	61 500	0.95	0.49
Forged Manganese Bronze	17 500	16 000	1.09	0.17

In Table II are given the endurance limits obtained for these various materials in direct stress. For the purpose of comparison, fatigue tests were made on the same material under cycles of flexural stress. These endurance limits are also given in Table II. The third column of the table gives the ratio of the endurance limit under direct stress to the endurance limit under flexural stress and shows this ratio to be approximately unity.

The fourth column gives the ratio of endurance limit to strength in tension—the steels agreeing closely in this respect. It will be observed that in this ratio the non-ferrous metal varies markedly from the ferrous.

CONCLUSIONS

From these considerations, it is concluded that a satisfactory attachment has been designed, for direct stress testing, which makes possible the speedy insertion of test specimens and their removal without injury to the surface of the fracture. Furthermore, a new method of calibration has been devised in conjunction with a new design of high-speed extensometer which gives accuracy in calibration and which may have other applications.

A result of more importance to the designer, however, lies in the comparative data which shows (for all cases examined) that the endurance limit obtained by direct stress is the same as that obtained by flexural stress.

Acknowledgment.—The author wishes to thank the Westinghouse Electric and Manufacturing Co., in whose research laboratory this work has been executed, for permission to publish these results, and Messrs. J. M. Lessells and S. Timoshenko for valuable criticism and assistance in the preparation of the manuscript.

DISCUSSION

Mr. Moore.

MR. H. F. MOORE[1] (*presented in written form*).—Mr. Irwin's paper gives some very significant test results, which seem to bear out his conclusion that the endurance limit for a metal is practically the same for reversals of axial stress (tension-compression) as it is for reversals of flexural stress. This does not agree with some previously obtained test results from other laboratories, but probably the difference is due to the great care used by Mr. Irwin in obtaining pure axial stress on his specimens. Other experimenters, including the writer, have not taken such thorough precautions. It is very difficult indeed to get pure axial stress on a short specimen. It is to be hoped that Mr. Irwin's careful work may be extended and check results obtained.

One conclusion which may be drawn from Mr. Irwin's work—a conclusion very valuable to the testing engineer—is that the inexpensive rotating-beam fatigue testing machine gives reliable results for endurance limit. Ten rotating-beam machines can be purchased for the price of one machine such as Mr. Irwin used. The tension-compression machine has a field of usefulness in connection with certain research problems, but for the general determination of endurance limits Mr. Irwin's tests seem to show the reliability of the rotating-beam machine, and to answer certain sweeping criticisms of it, which have recently been made.

In connection with the difference between Mr. Irwin's results and those obtained by earlier experimenters with tension-compression machines it might be noted that these early experimenters secured as even a distribution of stress in their specimens as is secured in machine parts subjected to repeated axial load—such parts as piston rods and connecting rods. The difficulty of getting uniform stress distribution in a short tension or compression member has not been appreciated, either by testing engineers or by machine designers. The actual maximum stress on a piston rod is usually distinctly greater than the value obtained by dividing load by area of cross-section. Under static load this excess stress is usually not important because it is confined to a small area, but under repeated stress, such as occurs in the service of a piston rod, this excess stress is important because it may start a fatigue crack. In designing members to resist repeated

[1] Professor of Engineering Materials, University of Illinois, Urbana, Ill.

axial stress it seems necessary to allow a factor for stress-intensification which, in some cases, may be as high as 1.30 or 1.40. Mr. Moore.

MR. PAUL L. IRWIN (*author's closure by letter*).—The author appreciates the remarks which have been made by Mr. Moore. It is true that the rotating beam fatigue testing machine is comparatively inexpensive and gives reliable endurance limits. Mr. Irwin.

In considering the relative costs of the rotating beam and direct stress machines, it must be remembered that the latter lends itself to a study of the effect of varying the proportion of tension to compression in the stress cycle. In addition to this, the direct stress machine is of especial value due to its adaptability to high temperature fatigue tests—the prerequisite examination of metals at normal temperature giving rise to the present paper.

Tests are now in progress which utilize these features of the direct stress test and which, we believe, will supply additional information on the fatigue of metals.

SOME FATIGUE TESTS ON NON-FERROUS METALS

By R. R. Moore[1]

Synopsis

This paper gives the results of endurance tests on pure magnesium, aluminum, a forged and cast magnesium-aluminum alloy, and naval brass.

An improved type of rotating beam machine of wide range of use is also described.

In addition to the determination of the endurance limits, the ratio of endurance limit to other physical properties is developed. Special attention is given to the "endurance weight efficiency" and comparisons are drawn between light non-ferrous metals and alloy steels. The effect of a notch upon the endurance limit and the effect of ductility upon the notch effect is discussed. Endurance tests on certain of the light alloys run to 600,000,000 reversals of stress give evidence that there exists a definite endurance limit for some of the alloys of this class.

In this paper are given the results of tests on non-ferrous metals to determine their endurance limits when subjected to alternating stresses of tension and compression produced by the rotating beam method. The data given represent some results of an investigation of the fatigue of non-ferrous metals which is being conducted at the Engineering Division, U. S. Air Service, as part of a general investigation into the applicability of metals to aircraft construction. The tests were run in the Physical Testing Laboratory of the Engineering Division at McCook Field, Dayton, Ohio.

Materials

The metals tested are briefly as follows: Naval brass, rolled aluminum, extruded magnesium, and a magnesium-aluminum alloy. The details of chemical composition are given in Table I. The naval brass specimens were taken from rolled bars $\frac{3}{4}$ in. in diameter. The aluminum specimens were taken from rolled bars of commercial aluminum $\frac{3}{4}$ in. in diameter. The magnesium specimens were taken from extruded bars of practically pure magnesium, $\frac{1}{2}$ in. square in cross-section. The magnesium-aluminum alloy specimens were cut from an airplane propeller which was forged from a 7-in. billet. Specimens were taken from the hub and represent the material as cast,

[1] Chief of Physical Testing Branch, Material Section, Engineering Division, U. S. Air Service, McCook Field, Dayton, O.

and from the forged blade which represents the material as forged. In this latter case, specimens were taken both with and across the direction of working.

There are also reported some long-time tests of three extruded magnesium alloys.

In order to record the structure of the different metals, micrographs were taken and are shown in Fig. 1. A comparison of micrographs (*a*), (*b*), and (*c*) of the magnesium-aluminum alloy propeller shows that the hub of the propeller suffered practically no working, while the blade was considerably worked both in a longitudinal and transverse direction.

GENERAL TESTS FOR PHYSICAL PROPERTIES

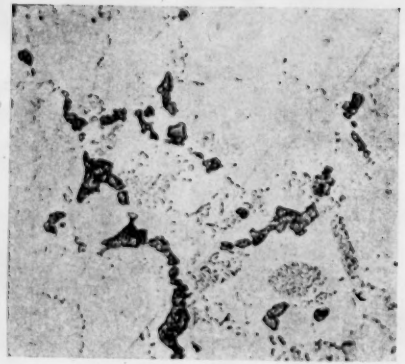
Tension, compression, torsion, shear, impact (Izod and Charpy notched bar), hardness (Rockwell and Brinell), and specific gravity tests were made. The method of procedure and test specimens used in these tests have been previously described(1).[1]

TABLE I.—CHEMICAL COMPOSITION OF METALS TESTED.

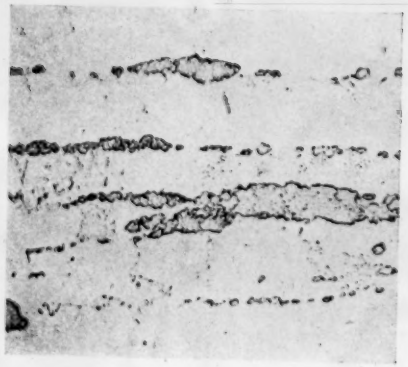
	VALUES IN PER CENT							
MATERIAL	COPPER	ZINC	LEAD	TIN	ALUMINUM	IRON	SILICON	MAGNESIUM
Naval Brass	61.20	Rem.	0.10	0.43	...	...	...	...
Magnesium-Aluminum Alloy	0.026	...	...	...	8.68	0.041	0.023	Rem.
Aluminum	0.12	...	...	...	Rem.	0.49	0.15	...
Magnesium	...	...	...	...	...	0.02	0.02	Rem.

Results of Tests.—The results of these tests are given in Table II. It is seen that magnesium, which is 0.79 as heavy as aluminum, has a tensile strength 1.44 times as high. However, attention is called to its extremely low proportional limit and ductility. The alloy of magnesium and aluminum (forged-longitudinal) is only slightly heavier than pure magnesium, and has a tensile strength 1.27 times that of magnesium and 1.82 times that of aluminum. The proportional limit of the forged alloy, however, shows a remarkable increase over that of pure magnesium; it is practically 9.3 times as high. It is interesting to note the difference between the proportional limit and tensile strengths of the alloy as cast, and forged (longitudinal). There is also considerable difference between the proportional limit and tensile strength of the forged material when tested with and across the direction of working. Naval brass, which has 4.7 times the weight

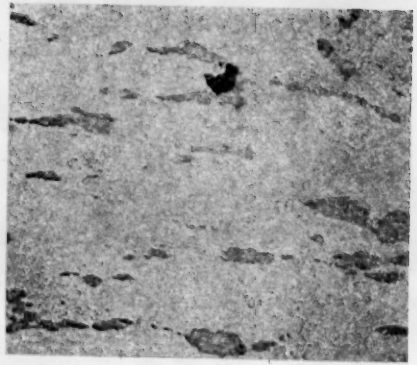
[1] The boldface numbers in parentheses refer to the papers given in the list of references appended hereto.



(*a*) Hub (Longitudinal) Etching Solution, 0.5 per cent HF 5 Seconds (×500)

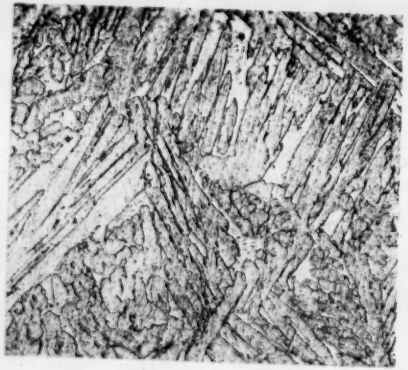


(*b*) Blade (Longitudinal) Etching Solution, 0.5 per cent HF 5 Seconds (×500)

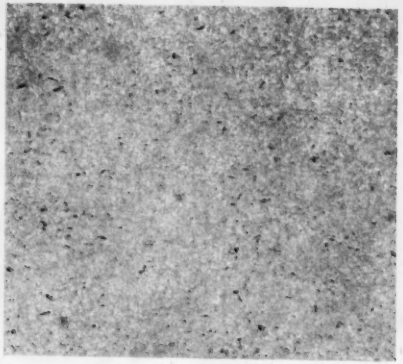


(*c*) Blade (Transverse) Etching Solution, None (×500)

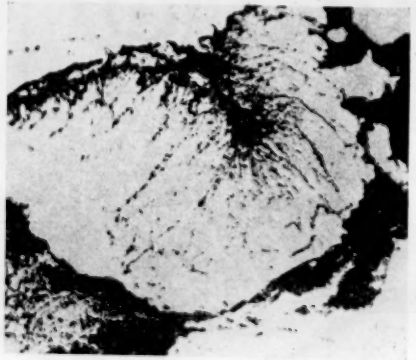
Magnesium-Aluminum Alloys.



Naval Brass (Transverse) Etching Solution, $NH_4OH + H_2O$ (×100)



Aluminum (Transverse) Etching Solution, None (×100)



Magnesium (Transverse) (showing inclusions) Etching Solution, None (×500)

FIG. 1.—Micrographs of Metals Tested.

of the forged alloy, has a proportional limit only 2.93 times as great and a tensile strength of 1.65 times as great. The shearing strength of the alloy as cast is greater than the shearing strength as forged or than that of the extruded magnesium or aluminum. The resistance of the alloy or the pure magnesium to shock as measured by the notched-bar impact test is evidently very low.

TABLE II.—PHYSICAL PROPERTIES OF METALS TESTED.

	Aluminum	Magnesium	Magnesium-Aluminum Alloy			Naval Brass
			Forged (Longitudinal)	Forged (Cross Grain)	As Cast	
TENSION						
Proportional Limit, lb. per sq. in...	11 275	1 224	11 415	7 433	6 875	33 415
Tensile Strength, lb. per sq. in.....	22 600	32 490	41 275	30 215	27 950	68 215
Elongation, per cent.....	16.0	6.2	4.0	3.0	4.0	27
Reduction of Area, per cent.....	65.4	4.4	None	None	None	53
Modulus of Elasticity, lb. per sq. in.	10 133 000	6 736 000	6 387 000	6 027 000	5 910 000	15 262 000
COMPRESSION						
Proportional Limit, lb. per sq. in...	11 430	5 730	20 370	....	....	32 300
Compressive Strength, lb. per sq. in.	25 630	34 580	41 860	46 730	40 000	72 790
SHEARING STRENGTH, LB. PER SQ. IN.	13 795	15 513	19 793	17 910	20 640	45 490
TORSION						
Proportional Limit, lb. per sq. in...	6 150	2 725	5 360	....	3 118	25 700
Modulus of Rupture, lb. per sq. in..	18 140	22 810	24 710	22 650	24 340	60 600
Modulus of Elasticity, lb. per sq. in.	3 451 000	577 000	2 332 000	....	2 442 000	5 396 000
IMPACT						
Charpy, ft-lb.....	18.72	2.90	1.8	1.35	1.53	24.6
Izod, ft-lb.....	24.0	4.0	4.0	2.33	2.50	33.9
HARDNESS						
Brinell, 500 kg.....	45	41	61	....	61	13.5
Rockwell, $\frac{1}{8}$-in. ball.....	23	14	45	....	40	73[a]
SPECIFIC GRAVITY.....	2.190	1.728	1.785	....	1.795	8.423
ENDURANCE LIMIT, LB. PER SQ. IN...	10 500	7 800	15 000	13 000	12 500	21 000
ENDURANCE LIMIT OF NOTCHED SPECIMEN, LB PER SQ. IN.....	4 000	3 000	7 000	7 500	7 000	13 500
REDUCTION OF ENDURANCE LIMIT DUE TO NOTCH, PER CENT.....	63	61.5	53	42	44	35.5

[a] $\frac{1}{16}$-in. ball.

ALTERNATING STRESS TESTS

Testing Machines and Test Specimens.—The alternating stresses produced in the test specimens were reversals of stress of equal magnitude from tension to compression. The stress is not evenly distributed over the cross-section of the specimen but varies from zero at the center to a maximum at the outer surface. The endurance limits, therefore, are extreme fiber stress calculated by means of the bending formulas according to the elastic theory and are subject to all the limitations which the use of this formula involves.

The machine used to produce these stresses is the two-point symmetrically loaded rotating beam machine, which is often referred

Etching Solution, None (×500)
Fig. 1.—Micrographs of Metals Tested.

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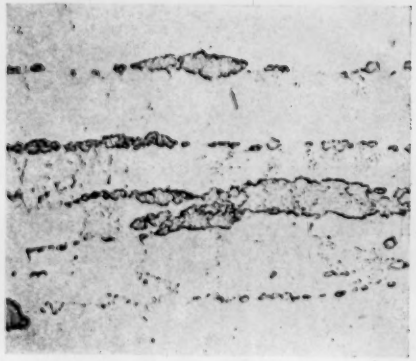
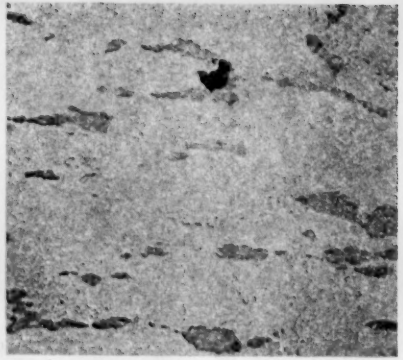


(*b*) Blade (Longitudinal) Etching Solution, 0.5 per cent HF 5 Seconds (×500)



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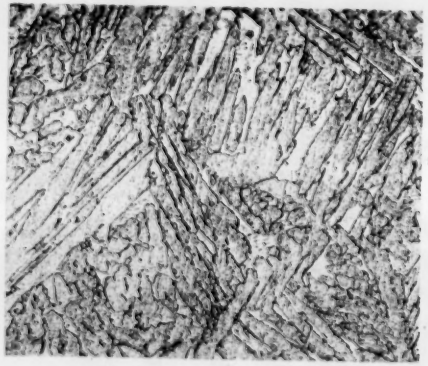

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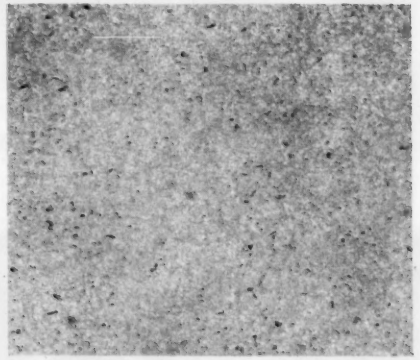
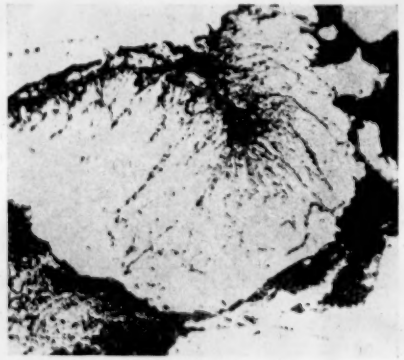


Aluminum (Transverse) Etching Solution, None (×100)



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Testing Machines and Test Specimens.—The alternating stresses produced in the test specimens were reversals of stress of equal magnitude from tension to compression. The stress is not evenly distributed over the cross-section of the specimen but varies from zero at the center to a maximum at the outer surface. The endurance limits, therefore, are extreme fiber stress calculated by means of the bending formulas according to the elastic theory and are subject to all the limitations which the use of this formula involves.

The machine used to produce these stresses is the two-point symmetrically loaded rotating beam machine, which is often referred

to as the Sondericker or Farmer type. The writer has previously(1, 2) called attention to the difficulty of testing light metals such as aluminum or magnesium and their alloys with rotating beam machines of the long-specimen, ball-bearing type. Two types of machines, one a plain bearing machine and the other a ball-bearing machine employing a short specimen, have been described(1). Further development has produced a machine which combines the advantages of both these types in that it has plain bearings and employs a short specimen. This machine was used throughout these tests and is shown in Fig. 2. Its peculiar advantages are that it uses an inexpensive type of specimen, is easily and quickly set up, runs practically without noise or vibration, and is very light so that extremely low stresses can be produced. By its use considerable time and expense in the testing of light alloys can be saved due to the advantage of having short, stiff specimens which do not become bent in the turning and polishing

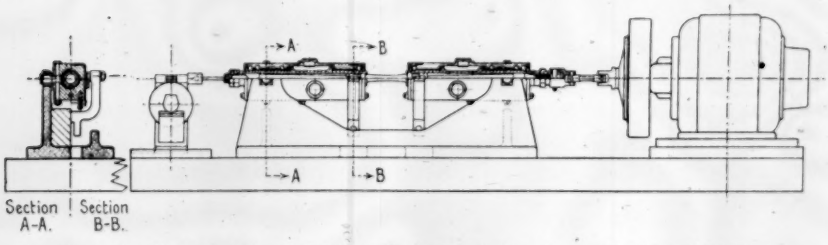


FIG. 2.—Short Specimen Plain Bearing Rotating Beam Fatigue Machine.

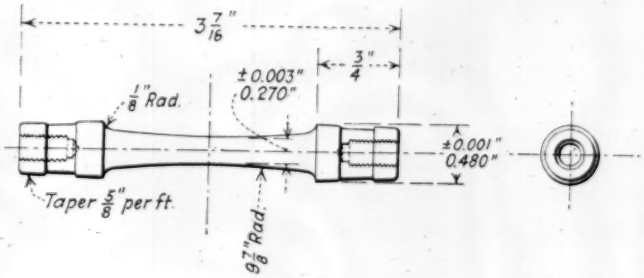
operations, or in setting up. In fact, with this machine a test can be stopped and the specimen removed and later replaced without any danger of deforming it. The machine is not limited to testing soft metals; it can also be used to test hardened steels. During the coming year machines of this type are to be used on tests of welds. In the testing of metals across the direction of working much less material is available for making test specimens so that some sort of short-specimen machine is necessary.

The test specimen used is shown in Fig. 3 (*a*). It will be noticed that the radius of the reduced section is the same as formerly employed on the long-type specimens. All test specimens were polished at the reduced section in order to eliminate tool marks and scratches. Polishing was done by applying to the rotating specimen several grades of Manning paper (in order No. 2, 1, 0, 00, 000) and No. 65 levigated alumina.

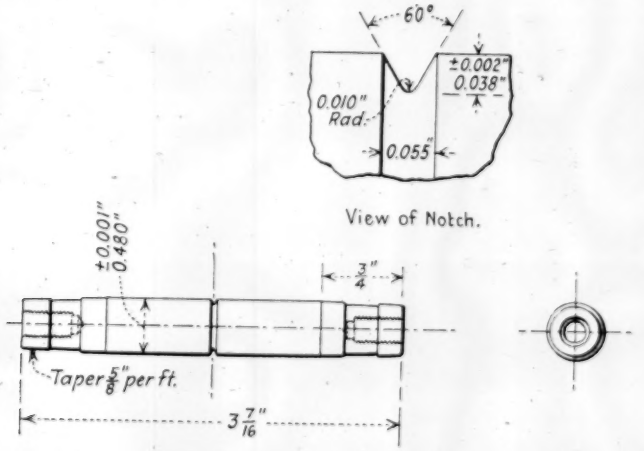
The speed of tests was 1750 r.p.m.

Results of Tests.—The results of endurance tests are shown in Figs. 4 to 7, inclusive, which are plotted to semi-logarithmic coordinates. The actual values of the endurance limits are given at the end of Table II.

Pure magnesium has a lower endurance limit than aluminum, although it has a much higher tensile strength. This may be some-



(*a*) Plain Specimens.



(*b*) Notched Specimens.

FIG. 3.—Fatigue Specimens.

what influenced by the extremely low proportional limit of magnesium. It is evident, however, from the other test results shown here that the low proportional limit is not alone responsible for the poor endurance limit value. For instance, the cast alloy which has a proportional limit of only 60 per cent of that of aluminum has an endurance limit 20 per cent higher. Likewise, the naval brass has a proportional limit

practically three times as great as the forged light alloy (longitudinal) but has an endurance limit only 40 per cent greater. Evidently, the influence of proportional limit is a minor factor.

The ratios of endurance limit to other physical properties, given in Table III, are seen to be quite variable. The ratio of endurance limit to tensile strength, which is about 0.50 for most ferrous metals, is evidently much lower for the non-ferrous metals. The ratio for magnesium is very low, while the ratio for aluminum approaches that of the ferrous metals. It is interesting to note that the ratio for the magnesium-aluminum alloy is very much higher than the ratio for pure magnesium even though the alloy is mostly magnesium. The addition of 8.5 per cent of aluminum evidently enhances this ratio considerably.

To the designer of airplanes weight is a vital consideration. It is also a matter of great importance in the construction of equipment for

TABLE III.—RATIO OF ENDURANCE LIMIT TO PHYSICAL PROPERTIES.

Ratio of Endurance Limit to:	Aluminum	Magnesium	Magnesium-Aluminum Alloy			Naval Brass
			Forged (Longitudinal)	Forged (Cross Grain)	As Cast	
Proportional Limit (Tension)	0.931	6.37	1.315	1.75	1.818	0.628
Tensile Strength	0.465	0.24	0.363	0.43	0.447	0.308
Compressive Strength	0.41	0.226	0.358	0.278	0.312	0.288
Shearing Strength	0.761	0.503	0.756	0.726	0.605	0.462
Proportional Limit (Torsion)	1.705	2.86	2.80	....	4.08	0.817
Modulus of Rupture (Torsion)	0.579	0.342	0.607	0.574	0.514	0.346
Brinell Hardness × 1000	0.233	0.1902	0.246	....	0.205	0.155
Specific Gravity × 1000	4.79	4.51	8.40	7.28	6.96	2.49

other modes of transportation. The advantages of light, durable construction will be recognized by builders and users of automobiles, freight cars, shipping containers, etc. The reduction of useless excess weight means a reduction in operating and maintenance costs and an increase in live-load carrying capacities. It is most interesting, therefore, to compare the endurance limit with the specific gravity of the metal. Such a relation might be called the "endurance-weight efficiency." Table III shows that the forged light alloy gives an efficiency of 8.40, which is greater than that of any of the other metals listed and is about 1.75 times as great as the efficiency of either of its components. Attention is directed to the extremely low endurance-weight efficiency of naval brass. The author has given elsewhere(2) the results of some tests on electron metal (a magnesium-zinc alloy) and duralumin. The endurance-weight efficiency for electron is 9.56, which is higher than that of the magnesium-aluminum alloy given here. Duralumin, based on an endurance limit of 14,000 lb. per

sq. in., would have an efficiency of 5.0, so that in this respect it is clearly much inferior to the magnesium-aluminum alloy and especially to the magnesium-zinc alloy.

In Table IV are given some comparisons of the endurance-weight efficiencies of various metals. All the heavy non-ferrous metals show quite low efficiencies. Some very interesting points are brought out by comparing the light alloys with the steels. A plain cold-drawn carbon steel with a tensile strength of about 75,000 lb. per sq. in. has a ratio of about 4.59. This is not quite as good as duralumin and very much below the magnesium-aluminum alloys. A cold-drawn 3.5-per-cent nickel steel (S. A. E. No. 2330), which might run about 100,000 lb. per sq. in. tensile strength, has a value of 6.12. This is better than

TABLE IV.—ENDURANCE-WEIGHT EFFICIENCY OF VARIOUS METALS.

ENDURANCE-WEIGHT EFFICIENCY IS RATIO OF ENDURANCE LIMIT TO SPECIFIC GRAVITY × 1000.

NON-FERROUS—LIGHT		NON-FERROUS—HEAVY	
Aluminum (Rolled)	4.79	Aluminum Bronze Heat Treated (Extruded)	4.50
Magnesium (Extruded)	4.51	Aluminum Bronze (Cast)	2.90
Mg. Base+4 per cent Al+¼ per cent Mn. (Extruded)	8.48	Aluminum Bronze Heat Treated (Cast)	3.44
Mg. Base+6.5 per cent Al+¼ per cent Mn (Extruded)	8.39	Manganese Bronze (Cast)	2.06
Mg. Base+8.5 per cent Al (Forged)	8.40	Naval Brass	2.49
Mg. Base+8.5 per cent Al (Cast)	6.96	FERROUS.	
Mg. Base+10 per cent Cu (Extruded)	5.79	Plain Carbon Steel (Cold Drawn)[1]	4.59
Mg. Base+4.5 per cent Zn. (Extruded)	9.56	3.5-per-cent Nickel (Cold Drawn)[2]	6.12
Duralumin (Rolled)	5.0	1.20-per-cent Carbon Steel[3]	13.65
		3.5-per-cent Nickel Steel[4]	15.30

[1] Tensile strength about 75,000 lb. per sq. in.
[2] Tensile strength about 100,000 lb. per sq. in.
[3] Heat treated to tensile strength of 224,000 lb. per sq. in.
[4] Heat treated to tensile strength of 294,000 lb. per sq. in.

duralumin but still not as good as the magnesium-aluminum alloy. A 1.20-per-cent carbon steel, heat-treated to a tensile strength of 224,000 lb. per sq. in., has a value of 13.65. This is much better than any of the light alloys. A 3.5-per-cent nickel steel, heat-treated to a tensile strength of 294,000 lb. per sq. in., has a ratio of 15.30. It is evident, therefore, that plain carbon or a cold-rolled alloy steel is not as efficient as some light alloys, but by heat treatment the alloy or high-carbon steels will yield a much greater efficiency.

Effect of "Coaxing."—Gillett and Mack(3) have termed the process of gradually increasing the stress during the endurance test as "coaxing." The process of coaxing was tried on the naval brass, aluminum

magnesium, and the magnesium-aluminum alloy. The results are indicated in Fig. 4 by the dotted lines which connect the steps of the process. With the naval brass, a specimen which had withstood 16,000,000 reversals at a stress of 22,000 lb. per sq. in. was stepped up to 24,000 lb. per sq. in. and failed after 2,000,000 reversals at that stress. It will be noticed that a specimen which was originally run at 24,000 lb. per sq. in. withstood over 4,000,000 reversals, so that it would appear that stressing at 22,000 lb. per sq. in. is detrimental rather than beneficial and it might be concluded that the stress of 22,000 lb. per sq. in. is above the endurance limit, which seems to be

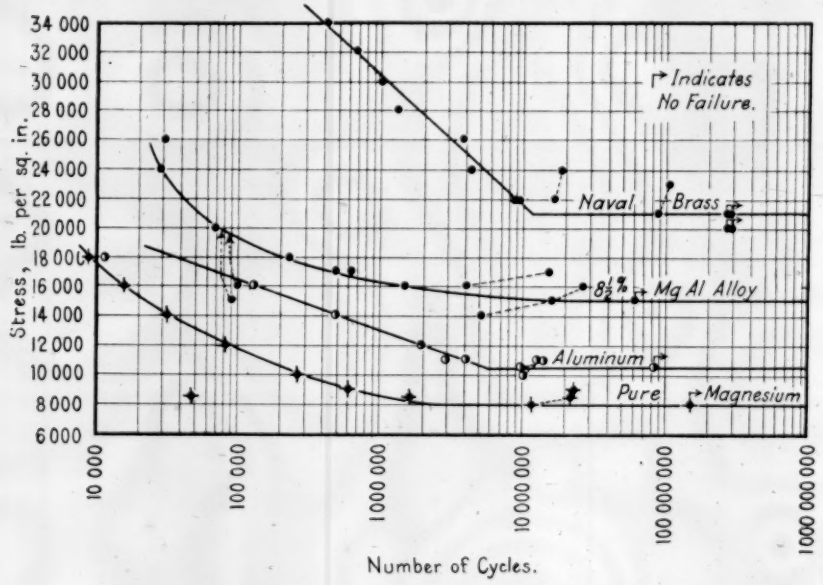


FIG. 4.—Endurance Tests on Plain Specimens.

the case. A specimen which had withstood 88,000,000 reversals at 21,000 lb per sq. in. was stepped up to 23,000 and withstood 15,000,000 reversals at that stress. The endurance curve shows that at 23,000 lb. per sq. in. the material should have a life of 7,000,000 reversals. Evidently, then, the stress of 21,000 lb. per sq. in. is not detrimental but beneficial, as it increases the life at the higher stress by 100 per cent.

A specimen of the magnesium-aluminum alloy, after withstanding 4,000,000 reversals at 16,000 lb. per sq. in., was stepped up to 17,000 lb. per sq. in. and stood 10,000,000 reversals before failure. The endurance curve shows the ordinary life of the material at that stress

to be 500,000 reversals, so it seems certain that stressing at 16,000 lb. per sq. in. has considerably strengthened the material. Another specimen which had withstood 5,000,000 reversals at 14,000 lb. per sq. in. was stepped up to 15,000 lb. per sq. in. and run for 10,500,000 reversals; then stepped up to 16,000 lb. per sq. in. where it finally failed after 10,000,000 reversals. The endurance curve would indicate the life of the material at 16,000 lb. per sq. in. stress to be 1,500,000 reversals. It is evident that the process of coaxing has considerably lengthened the life of the material (about 600 per cent). Attention is directed to the fact that the stress of 16,000 lb. per sq. in. strengthened the material although it is above the endurance limit, as indicated

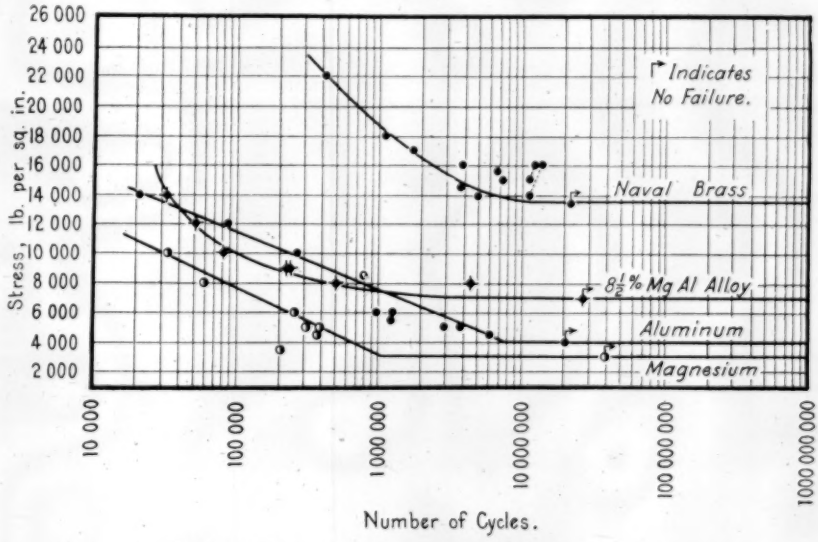


FIG. 5.—Endurance Tests on Notched Specimens.

in Fig. 4 (15,000 lb. per sq. in.). This may be accounted for by the irregularity in the material, the specimen discussed being an exceptionally good one while a previous specimen at the same stress falls on the curve, and another one has a very short life (100,000 reversals). There is also shown another specimen at 15,000 lb. per sq. in. which had a very short life.

Coaxing tests were also run on the aluminum. A specimen which had withstood 10,000,000 reversals at 10,500 lb. per sq. in. was stepped up to 11,000 lb. per sq. in. and failed at 3,250,000 reversals. Another specimen which had withstood 10,000,000 reversals at 10,000 lb. per sq. in. was stepped up to 11,000 lb. per sq. in. and failed after 4,000,000 reversals. There is indication here of a slight increase in the life of

the material, but it appears that for aluminum the coaxing must be done at lower stresses to obtain beneficial results of any account.

A coaxing test on magnesium was started at 8000 lb. per sq. in. at which it resisted 11,500,000 reversals and was then stepped up to 8500 lb. per sq. in. for 10,500,000 reversals and then to 9000 lb. per sq. in., at which it failed after 500,000 reversals.

The net result of these tests seems to indicate that stressing at or below the endurance limit is beneficial to the naval brass, pure magnesium, and the alloy. Aluminum, although it showed no harmful effects, did not show a decided improvement at the stresses experimented with.

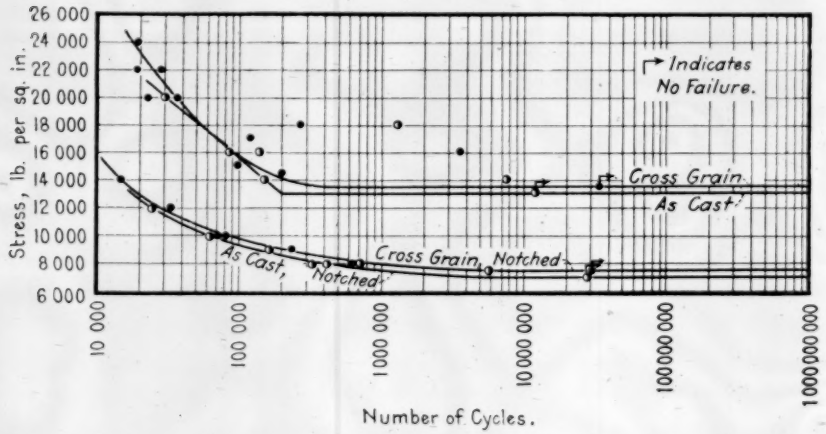


FIG. 6.—Endurance Tests on Cast, Cross-Grain, and Notched Magnesium Alloy.

Moore and Jasper(6) have from their tests on ferrous metals described the action of repeated stress as being of both a beneficial and destructive nature, depending upon the value of the stress. McAdam(7) has shown that nickel, monel metal, copper, copper-nickel, copper-nickel-aluminum alloys, aluminum bronze, and manganese bronze also respond to coaxing. The author has shown elsewhere similar results on manganese bronze(1). The tests described in the present paper would indicate that some light non-ferrous alloys also are affected both beneficially and adversely, although the possibilities of improvement by understressing are not as great as with the heavy non-ferrous alloys and with the steels. These tests do not demonstrate that the value of the endurance limit can actually be raised as has been done on steels, but only that there is a possibility of improvement by understressing. However, these tests are far too few to

draw the conclusion that this cannot be accomplished by a modification in the process.

Effect of Direction of Grain.—The specimens taken from the blade of the light alloy airplane propeller were tested with and across the direction of working. The results of the cross-grain tests, shown in Fig. 6, are rather erratic, probably due to the influence of inclusions and laminations. The endurance limit found on the cross-grain specimens was 13,000 lb. per sq. in., which is about 13 per cent less than for the longitudinal specimens. This is not a great depreciation, but does show that the material is less reliable across the direction of working. The most serious factor to be considered here is not the actual reduction in endurance limit but the irregularity of the material.

Other specimens were cut from the hub and represent the material in the "as cast" condition, as this part of the propeller was turned from a 7-in. ingot and received no working. The points on the endurance curve of the "as cast" material show considerable scatter, which indicates irregularity in the material. The actual value of the endurance limit is 12,500 lb. per sq. in., which is only about 16.5 per cent lower than the forged material with the grain. This high relation shows very promising for cast light alloys. Again it seems to the author that the most important consideration is not the lower endurance limit but the irregularity of the material.

Effect of Notch.—The effect of the presence of a notch upon the endurance limit of a metal is not as great as would be suspected from the value of the concentrated stresses caused by the notch as determined mathematically or by the photo-elastic or soap film methods. The author has shown elsewhere(1) that if it be assumed that the stress in the specimen should nowhere exceed the endurance limit, then the notch shown in Fig. 3 (*b*) should cause a reduction in endurance limit of about 78 per cent. Tests were run on specimens containing such a notch and the results are plotted in Figs. 5 and 6. In the last three lines of Table II are given the endurance limits obtained from these curves and the percentage reduction in endurance limit due to the notch.

The reduction in endurance limit is greater on the pure metals than it is on the alloy of both, and is less on the cast alloy than on the forged alloy (longitudinal). The cast material has an endurance limit when notched which is equivalent to the longitudinal forged material. This is certainly an interesting result and shows that the cast alloy has very good endurance properties even under severe conditions and despite a low proportional limit and tensile strength and is, therefore, more useful than is commonly supposed. An examination of Table II

also will show that the impact resistance of the cast alloy is slightly better than that of the cross-grain forged material and not much less than the longitudinal forged material. Elsewhere(1) the author has shown that a 4-per-cent extruded alloy of magnesium and aluminum suffers a reduction in endurance limit of about 33 per cent, which should be compared with the 53 per cent for the 8.5-per-cent alloy reported here. The difference of the effects of extruding and forging may account for some of the discrepancy between the results.

TABLE V.—COMPARISON OF EFFECT OF NOTCH ON ENDURANCE LIMIT WITH ELONGATION OF MATERIAL.

Material	Elongation, per cent	Reduction of Endurance Limit, per cent	Type of Notch	Authority
Armco Iron (Rolled)	48.3	15	$\frac{1}{4}$-in. Radius	Moore and Kommers(4)
0.49-per-cent Carbon Steel (Rolled)	23.5	8	"	" " "
Armco Iron (Rolled)	48.3	48	Square Shoulder	" " "
0.49 Carbon Steel (Rolled)	23.5	51	"	" " "
Mg+4 per cent Al (Extruded)	21.7	33	60-deg. V, 0.01 in Radius	R. R. Moore(1)
Mg+4 per cent Al+0.25 per cent Mn (Extruded)	15.5	33	See Fig. 3(*b*)	"
Mg+6.5 per cent Al (Extruded)	16.0	23	"	"
Mg+8.7 per cent Al (Forged-L)	4.0	53	"	Present paper
Mg+8.7 per cent Al (Forged-X)	3.0	42	"	R. R. Moore(1)
Mg+8.7 per cent Al (Cast)	4.0	44	"	"
Mg+10 per cent Cu (Extruded)	3.0	45	"	"
Magnesium (Extruded)	6.2	61.5	"	Present paper
Aluminum (Rolled)	16.0	63	"	" "
Naval Brass, heat-treated	27.0	35.5	"	" "
Extruded Al-Bronze, heat trested	35.5	41	"	R. R. Moore(1)
Cast Al-Bronze, heat treated	14.0	15.0	"	"
S. A. E. No. 1050 Steel (Rolled)	24.0	58	"	"

Much has been said about the value of the property of ductility as measured by elongation. The ability of a metal to relieve itself of the effects of concentrated stresses when subjected to alternating stresses has also been ascribed to this property. A careful consideration of the action occurring under repeated stressing and an examination of some test results on notched specimens will show that elongation, while it may contribute somewhat to the enhancement of the endurance of a notched specimen, is certainly not the controlling factor.

Table V was prepared from some test results given by Moore and Kommers(4) and from the author's own work. Comparing the tests on Armco iron and 0.49-per-cent carbon steel, it is seen that despite the fact that Armco iron has practically twice the elongation of the steel the effect of a notch of $\frac{1}{4}$-in. radius is almost twice as severe on the Armco iron. When the notch is a square shoulder the effect is to reduce the endurance limit about the same per cent in both cases. Considering only the magnesium alloys, it is seen that those alloys with extremely low elongation are more seriously affected than those

TABLE VI.—MINIMUM NUMBER OF REVERSALS OF STRESS TO DETERMINE ENDURANCE LIMIT OF SOME NON-FERROUS METALS.

Material	Number of Reversals	Authority
Nickel	300 000 000, No endurance limit reached	McAdam(7)
Monel (Cold Rolled)	200 000 000, " " " "	"
Monel Metal	300 000 000, " " " "	Moore and Jasper(5)
Monel (Hot Rolled)	200 000 000, " " " "	McAdam
Cu-Ni-Sn Alloy	20 000 000, " " " "	"
Copper	50 000 000, " " " "	"
Aluminum Bronze	20 000 000, " " " "	"
Brass	100 000 000, " " " "	Moore and Jasper(5)
Naval Brass	11 000 000, Endurance limit obtained	Fig. 4, present paper
Cast Al Bronze	1 000 000, 3 000 000, " " "	R. R. Moore(1)
Cast Mn Bronze	10 000 000, " " "	"
Duralumin	400 000 000, No endurance limit obtained	R. R. Moore(2)
Electron Metal	28 000 000, Endurance limit obtained	"
Mg-Al Alloy	500 000, " " "	Fig. 7, present paper
Aluminum	6 000 000, " " "	Fig. 4, " "
Magnesium	3 000 000, " " "	Fig. 4, " "

with a higher elongation. But compare with these results the reduction in endurance limit of 63 per cent of aluminum, which is a ductile material. The highest ductility shown in this table is 35.5 per cent obtained on the extruded aluminum bronze. Compare also its 41 per cent reduction with the reduction in endurance limit for the low ductility alloys. The cast aluminum bronze has an elongation of 14 per cent as compared to the 35.5 per cent of the same metal extruded, yet its reduction in endurance limit is 15 per cent as compared to the 41 per cent of the extruded metal.

These results show quite clearly that the notch effect is influenced but very little by elongation. When it is considered that failure under

repeated stress occurs with practically no elongation, it does not seem reasonable to expect elongation, which is a result of a continuous test to fracture caused by a gradually increasing stress which ultimately greatly exceeds the endurance limit, to have any effect upon that value. Even in repeated stress tests at very high stresses hardly any deformation is found after fracture.

EXISTENCE OF ENDURANCE LIMIT FOR NON-FERROUS METALS

It has been quite firmly established by H. F. Moore that there is a stress or range of stress at which the wrought ferrous metals have an exceedingly long life, probably indefinitely long. He has further shown from a large volume of careful tests that a steel specimen which

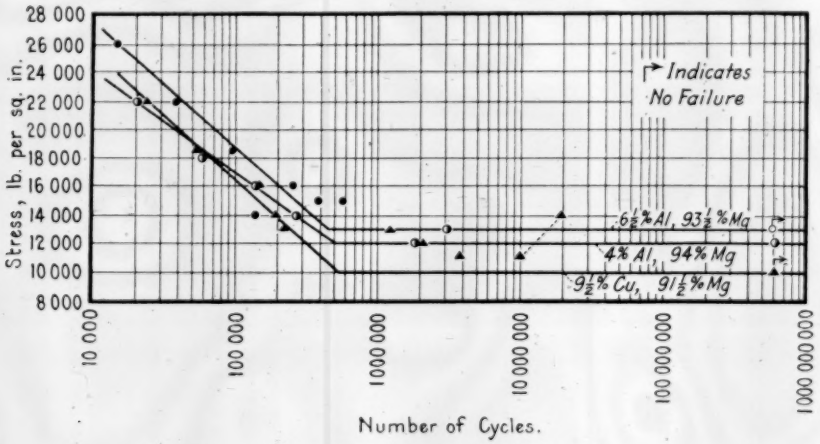


FIG. 7.—Long-Time Tests on Light Alloys.

will withstand 10,000,000 reversals of stress will have an exceedingly long life. Therefore, on steels the knee of the endurance curve is reached at or before 10,000,000 reversals. Data from tests on non-ferrous metals show that this latter conclusion cannot be applied to all non-ferrous metals. Table VI gives the results of tests by different investigators and indicates that in some cases a definite endurance limit is obtained below 10,000,000 reversals. In other cases, namely, nickel, monel metal, and duralumin, no endurance limit is obtained even at 300 and 400 million reversals. In still other cases, endurance limits are obtained between 10 and 100 million reversals.

The results of tests run in this laboratory on duralumin which indicated that this light alloy did not approach its endurance limit below 400,000,000 reversals of stress, led to some long-time tests on

the extruded magnesium alloys, the results of which are given in Fig. 7. It will be seen that these tests have run to 600,000,000 reversals. From the shape of the curves and the length of the test it appears that the specimens have reached an endurance limit or at least that they are going to have an exceedingly long life. These long-time tests are evidence that some non-ferrous metals may have an endurance limit as definite as is found with steels. A similar conclusion has also been reached by McAdam(8).

Attention is called to the shapes of the endurance curves for magnesium and for aluminum given in Fig. 4. The upper part of the aluminum curve is decidedly a straight line whereas the magnesium shows a curve. This means that the life of the magnesium specimens increases faster with decrease in stress than that of the aluminum specimens. The degree of this curve can be altered by changing the scale of plotting, especially the ordinate scale. If plotted to log-log coordinates this curve becomes practically a straight line. However, there does seem to be a difference in the endurance characteristics of these two metals. The curve of the magnesium-aluminum alloy follows the shape of the magnesium curve, which might be expected as the alloy consists mostly of magnesium.

THE ENDURANCE LIMIT AS A BASIS OF DESIGN

The high endurance-weight efficiency of the light alloys makes their use very desirable for many types of construction. The endurance-weight efficiency of course is not the sole basis for selection of a material. The comparative cost of the heat treatment of steels and the price of the non-ferrous alloys is an important consideration. Such initial cost, however, would be offset by lower maintenance costs, longer life, and higher live load carrying capacities. Another point to be considered is that the light metals require larger cross-sections due to lower allowable working stresses. These larger cross-sections in turn offer greater stiffness to bending and torsional deformation due to the larger section modulus. On the other hand, the lower modulus of elasticity of the light alloys affects this stiffness adversely. The selection of materials is a careful balancing of the factors mentioned above and many others depending upon the special use to which the part is put.

Due consideration of the endurance limit has been greatly neglected in design work. The designer of a machine usually applies a factor of safety to the value of the ultimate strength of the material. Occasionally, it is applied to the proportional limit. Parts of a

machine, however, practically never fail under a single application of a gradually increasing load as occurs in a testing machine when the strength of a material is being determined. On the contrary, it fails after a number of applications of stress. The endurance limit as a design basis is, therefore, a more reasonable approach to actual service conditions than the strength.

The rather constant ratio of endurance limit (rotating beam) to tensile strength which obtains on wrought ferrous metals is a lucky coincidence with the designer's selection of the strength as a basis for design. On non-ferrous metals and especially the light alloys the relation of endurance limit to proportional limit and tensile strength is erratic. In many cases the relation to tensile strength is dangerously low. The designer who uses non-ferrous metals as primary members in machines and structures should know the value of the endurance limit of the metal he is using as definitely as he knows the proportional limit and tensile strength. In those cases of the non-ferrous metals where the endurance limit exceeds the proportional limit the latter should of course take preference, as it is necessary to prevent permanent deformation of the structure.

CONCLUSIONS

1. The ratio of endurance limit to proportional limit and tensile strength for non-ferrous metals is quite erratic.

2. On a weight basis some light alloys are more efficient as regards endurance properties than either cold-rolled plain carbon or alloy steels but not as efficient as some heat-treated alloy steels.

3. The light alloys do not appear to respond to "coaxing" as readily as do the steels.

4. The magnesium-aluminum alloy "as cast" gives a relatively high endurance limit compared to that of the same metal forged.

5. The reduction of endurance limit due to concentrated stresses caused by a notch is not as severe on the cast light alloy as on the forged light alloy. Specimens of pure magnesium and aluminum are affected more than the alloy of both. The notch effect seems to be independent of elongation.

6. Long-time tests (600,000,000 reversals) indicate that some extruded magnesium-aluminum alloys have a definite endurance limit.

7. Designers should know the values for the endurance limits of non-ferrous metals when these metals are to be used as primary members for the endurace limit is often found to be very low.

Acknowledgment.—The writer wishes to acknowledge the valuable work of Mr. C. F. McMahon in running of tests and Mr. H. Caminez of the Engine Design Branch of the Engineering Division in the development of the testing machine.

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DISCUSSION

Mr. McAdam.

MR. D. J. MCADAM, JR.[1] (*presented in written form*).—Table VI of Mr. Moore's paper seems to be intended to give a general survey of the results obtained in three laboratories in investigation of the endurance properties of non-ferrous metals. There is nothing in his description of Table VI to show that he has selected for his table the results of only six stress-cycle graphs out of more than seventy published graphs obtained at the Naval Engineering Experiment Station with a great variety of non-ferrous metals.

For each of the six metals that Mr. Moore has selected to represent the investigation at the Naval Engineering Experiment Station, the statement is made in the second column of Table VI: "No endurance limit reached." There is nothing in Mr. Moore's paper to show that these conclusions are his own and not the conclusions reached at the Naval Experiment Station. That the conclusions expressed in column 2 of Table VI are not those of the Naval Experiment Station will be evident from the following quotations taken from the summaries of a series of recent papers by the writer. (One of these papers is given by Mr. Moore as a reference.) "The investigation includes nickel, monel metal, constantan, cupro-nickel, several ternary alloys of nickel, copper, alpha bronze, alpha brass, aluminum bronze, Muntz metal, Naval brass and manganese bronze. . . . The results presented in stress-cycle graphs show that there is a definite endurance limit for each of these alloys."[2] "The stress-cycle graphs here shown give additional evidence that the rotating-cantilever endurance limits of the alloys of nickel and of copper are as definite as the endurance limits for steel."[3]

The "Number of reversals" (by which is evidently meant "number of cycles") listed in Table VI presumably means, for each metal that is not credited with an endurance limit, the maximum number of cycles to which the metal was subjected. And yet for three of the six metals selected to represent the investigation at the Naval Experiment Station the numbers given by Mr. Moore are decidedly incorrect. He lists for aluminum bronze 20,000,000 cycles, whereas the four

[1] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md. Discussion presented by permission of the Secretary of the Navy.

[2] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part I," *Transactions*, Am. Soc. Steel Treating, January, 1925.

[3] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part III," *Transactions*, Am. Soc. Steel Treating, May, 1925.

aluminum bronzes of different composition tested at the Naval Experiment Station endured from about 72,000,000 to 125,000,000 cycles and then failed only after repeated increases of stress. (See Fig. 6 of paper by the writer.)[1] For a copper-nickel-tin alloy, instead of the 20,000,000 cycles listed in Table VI the number should be 55,000,000.[1] For copper, instead of 50,000,000 the number should be 63,000,000.[1] For all these metals and for many others a value for the endurance limit was given in the paper to which Mr. Moore has made reference. Mr. McAdam.

At the end of the first paragraph on page 81 is the statement that "some non-ferrous metals may have an endurance limit as definite as is found with steels." In the next sentence Mr. Moore asserts that the writer has expressed similar conclusions in a previous paper.[2] In that paper however, I find the following sentence: "Investigation of the endurance properties of non-ferrous metals indicates that for each non-ferrous metal there is an endurance limit that is practically as definite as the endurance limit for steel." The word "each" in this quotation has been replaced by the word "some" in Mr. Moore's reference to it.

In the last full paragraph on page 80, Mr. Moore says: "Table VI gives the results of tests by different investigators and indicates that in some cases a definite endurance limit is obtained below 10,000,000 reversals. In other cases, namely, nickel, monel metal and duralumin, no endurance limit is obtained even at 300 and 400 million reversals." And yet at least sixteen stress-cycle graphs, representing sixteen distinct samples of nickel tested at the Naval Experiment Station, all approach a horizontal direction at about 50,000,000 cycles. These graphs and estimated endurance limits, are given in recently published papers by the writer, one of which Mr. Moore has given as reference.[1, 3, 4]

Mr. Moore's conclusion that for duralumin no endurance limit is reached even at 400,000,000 cycles is based on his own published results, obtained with three samples of duralumin.[5] Two of these three graphs, however, if plotted on suitable scale, show definite evidence that an endurance limit has been reached at 50,000,000 to

[1] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part I," *Transactions*, Am. Soc. Steel Treating, January, 1925.

[2] D. J. McAdam, Jr., "Accelerated Fatigue Tests and Some Endurance Properties of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 454 (1924).

[3] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part II," *Transactions*, Am. Soc. Steel Treating, February, 1925.

[4] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper. Part III" *Transactions*, Am. Soc. Steel Treating, May, 1925.

[5] R. R. Moore, "Resistance of Manganese Bronze, Duralumin and Electron Metal to Alternating Stresses," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 106 (1923).

Mr. McAdam.

100,000,000 cycles. His third graph for duralumin is abnormal in form. Three of the four graphs published by Messrs. H. F. Moore and T. M. Jasper are of similar abnormal form.[1] These graphs of abnormal form have led Mr. R. R. Moore to conclude that duralumin and monel metal show no endurance limits even at 400,000,000 cycles.

The normal rotating-cantilever stress-cycle graph on a semi-logarithmic scale, for non-ferrous as well as for ferrous metals, is a curve descending with gradually decreasing slope to a horizontal or nearly horizontal tangent. The curvature varies with heat treatment, degree of cold working, and other factors. The "tangent" has a slight curvature but may be considered practically straight. The point of tangency varies from about 1,000,000 to 100,000,000 cycles. The abnormal graphs obtained by Mr. R. R. Moore[2] and by Messrs. H. F. Moore and T. M. Jasper[1] are nearly straight lines sloping downward throughout their entire length.

Typical stress-cycle graphs for ferrous and non-ferrous metals are given in a recent paper[3] by the writer. The following approximate quotation is from this paper: "Investigation at the Naval Experiment Station of a great variety of non-ferrous metals, including duralumin, monel metal, magnesium alloys, and all the commercial alloys of nickel and of copper has not disclosed any metal the endurance limit of which cannot be determined by individual tests of not more than 50,000,000 cycles. For some non-ferrous metals the stress-cycle graph evidently slopes slightly downward between abscissas of 50,000,000 and 100,000,000 cycles. When there is a slope between these abscissas, however, it is so slight that the ordinate does not vary more than about 2 per cent. Beyond an abscissa of 100,000,000 cycles, if there is any downward slope it is so slight that it can be disregarded in the determination of practical endurance limits."

It seems possible that the abnormal curves obtained by Mr. R. R. Moore[2] and by Messrs. H. F. Moore and T. M. Jasper[1] are due to some cause other than abnormality of material. This subject will be discussed in detail in a paper now in preparation. We should be pleased to make comparative tests at the Naval Experiment Station of any metal with which Mr. R. R. Moore or Messrs. H. F. Moore and T. M. Jasper have obtained stress-cycle graphs of abnormal form.

At this stage of progress, if a "normal" graph is being obtained, there is no advantage in running endurance tests beyond the point

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[1] H. F. Moore and T. M. Jasper, "An Investigation of the Fatigue of Metals," *Bulletin 142*, University of Illinois, XXI, No. 39, May 26, 1924.

[2] R. R. Moore, "Resistance of Manganese Bronze, Duralumin and Electron Metal to Alternating Stresses," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 106 (1923).

[3] D. J. McAdam, Jr., "Endurance Properties of Metals," *Mechanical Engineering*, Vol. 47, July, 1925.

of tangency of stress-cycle curve with a nearly horizontal line. In determining the point of tangency, and the form of the curve for a non-ferrous metal, much more information will be obtained by running six specimens each 100,000,000 cycles than by running one specimen 600,000,000 cycles. And still more information will be obtained by running twelve specimens each 50,000,000 cycles. Mr. McAdam.

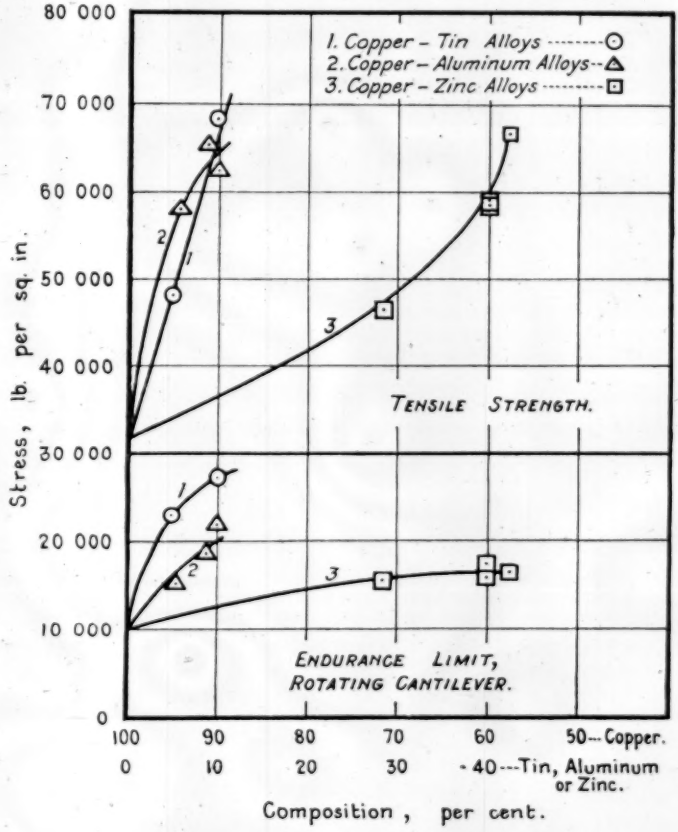

FIG. 1.—Effect of Proportions of Component Metals on Endurance Properties of Binary Alloys of Copper with Zinc, Aluminum and Tin.

About a year ago at the Naval Experiment Station we considered the evidence conclusive that non-ferrous metals have endurance limits as definite as those of ferrous metals. We then proceeded to investigate the influence of three variables affecting the endurance limits of non-ferrous metals, namely, heat treatment, cold working and chemical composition.

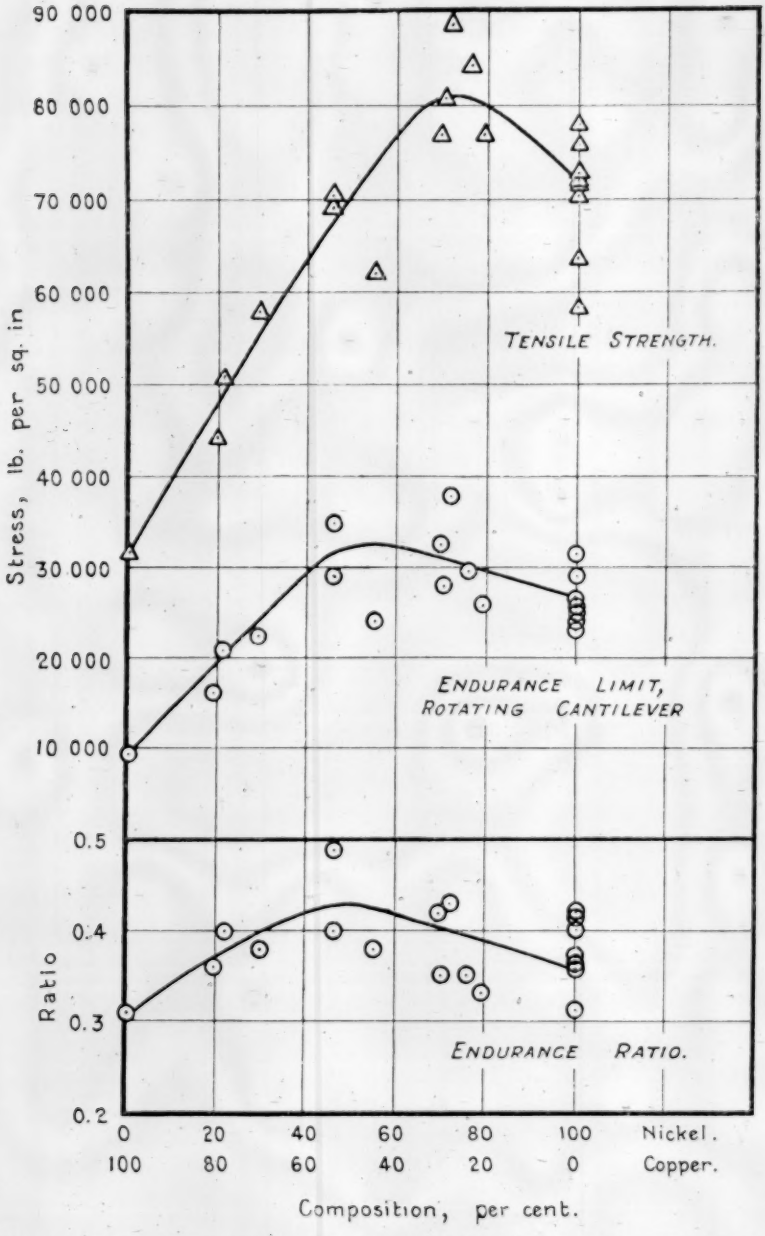

FIG. 2.—Effect of Proportion of Component Metals on Endurance Properties of Nickel-Copper Alloys.

Mr. McAdam.

Investigation of a number of alloys of nickel and of copper led to the conclusion that moderate cold working increases the rotating-cantilever endurance limit in proportion to the increase in tensile strength if internal stress be absent. Severe cold working, however, does not increase the rotating-cantilever endurance limit in proportion to the increase in tensile strength.[1, 2] In other words, the endurance ratio does not decrease with cold working until the boundary between moderate and severe cold working has been passed. This boundary has been determined for nickel[2] and is being investigated for other metals.

By means of this information the influence of one variable affecting the endurance limit of non-ferrous metals can be estimated. Endurance limits of fully annealed alloys can now be calculated from the endurance limits of the same alloys in the hot or cold-worked condition. By comparing the endurance limits of various alloys in the fully annealed condition the effect of chemical composition can be observed. Although other variables, such as grain size and quantity and distribution of non-metallic inclusions, affect the endurance limit, nevertheless it is now possible to draw curves illustrating the variation of the endurance limit with variation in the proportions of the two principal metals in a binary alloy. Such curves for binary alloys of copper with nickel, tin, zinc and aluminum have been presented by the writer in previous papers.[1, 2]

The accompanying Figs. 1 and 2 illustrate the effect of chemical composition on endurance limit and other physical properties. Some of these graphs are rearrangements of graphs presented in a recent series of papers by the writer.[1, 2, 3]

The upper three curves of Fig. 1 illustrate the effect of chemical composition on the tensile strength of binary alloys of copper with tin, aluminum and zinc, respectively. The lower three graphs represent for the same three series of alloys the effect of chemical composition on the rotating-cantilever endurance limit.

The lower curve of Fig. 1 shows the effect on the rotating-cantilever endurance limit of variation in the proportions of copper and zinc in binary alloys of these two metals. To represent the positions of alloys such as Naval brass and manganese bronze the composition of the equivalent binary alloy is calculated by means of Guillet's "coefficients of equivalence." The points representing endurance

[1] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part II," *Transactions*, Am. Soc. Steel Treating, February, 1926.

[2] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part III," *Transactions*, Am. Soc. Steel Treating, May, 1925.

[3] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part I," *Transactions*, Am. Soc. Steel Treating, January, 1925.

Mr. McAdam.

limits of alpha brass, Muntz metal, Naval brass and manganese bronze in the annealed condition all indicate a smooth curve that becomes practically horizontal at the limit of saturation of the alpha alloy (36 per cent of zinc). The value published by the Naval Experiment Station for endurance limit of Naval brass[1] is slightly lower than Mr. Moore's value. This is possibly due to the fact that his alloy was not so near to the fully annealed condition as the alloy used at the Naval Experiment Station. As stated in a previous paper,[1] the value for rolled manganese bronze is slightly higher than Mr. Moore's value for cast manganese bronze; this is due to the slightly greater tensile strength of the rolled alloy.

The upper and middle curves of the lower space of Fig. 1 illustrate the effect on the rotating-cantilever endurance limit of variation in the proportions of tin in the copper-tin alloy and aluminum in the copper-aluminum alloy, respectively. The approximate saturation limits of the copper-tin, copper-aluminum and copper-zinc alloys, respectively, are 13, 7 and 35 per cent.

The nickel-copper alloys form an unbroken series of solid solutions. If endurance limit be plotted against proportions of nickel and copper a smooth curve is obtained by reaching a maximum for alloys containing between 45 and 70 per cent of nickel. Such a curve taken from previous papers by the writer,[2, 3] is shown in Fig. 2. In this figure, the middle curve shows the effect of composition on the rotating-cantilever endurance limit. The upper and lower curves respectively illustrate the effect of composition on tensile strength and "endurance ratio." On these curves, monel metal falls into its proper place with the other nickel-copper alloys.

Mr. Moore.

MR. R. R. MOORE (*by letter*).—Mr. McAdam has evidently misinterpreted the significance of Table VI. The table is not intended as a general survey of the work of several laboratories, but to point out certain specific results obtained at these laboratories.

The notations in the second column of Table VI are most certainly the author's own conclusions, and it is desirable to emphasize the fact that these conclusions do not agree with those of Mr. McAdam as he quotes from his first two references. In order to determine whether or not Mr. McAdam has actually obtained endurance limits for the metals in question, it will be necessary to examine his graphs.

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[1] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part I," *Transactions*, Am. Soc. Steel Treating, January, 1925.

[2] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part II," *Transactions*, Am. Soc. Steel Treating, February, 1925.

[3] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part III," *Transactions*, Am. Soc. Steel Treating, May, 1925.

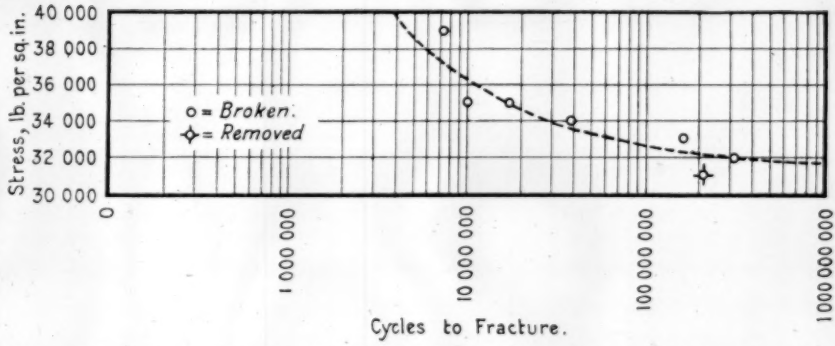

FIG. 3.—Curves Showing Rotating Cantilever Results on Annealed Nickel (Material A W 2).

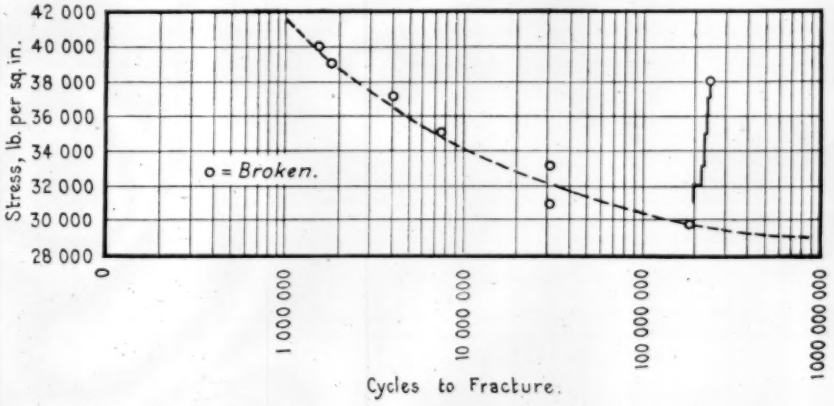

FIG. 4.—Curve Showing Rotating Cantilever Results on Monel Metal, Cold Rolled (Material BK).

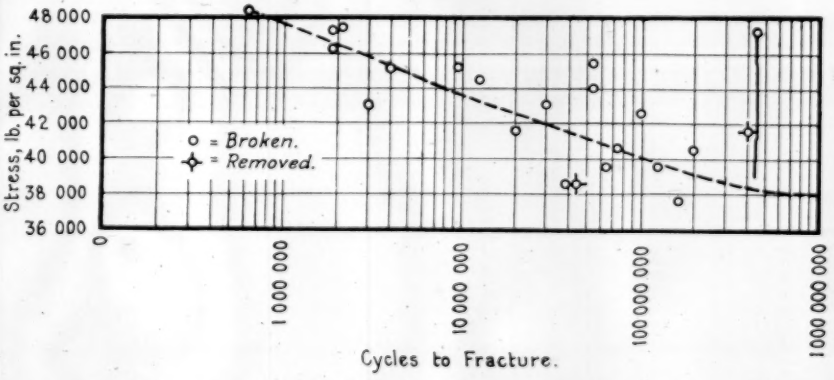

FIG. 5.—Curve Showing Rotating Cantilever Results on Monel Metal, Hot Rolled (Material A-A).

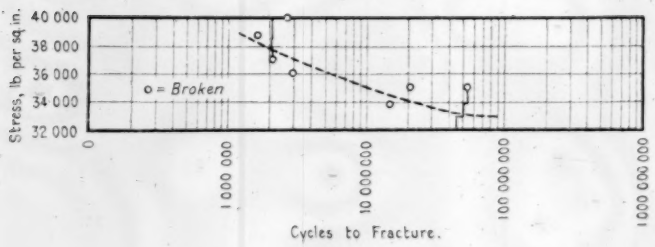

FIG. 6.—Curve Showing Rotating Cantilever Results on Copper-Nickel-Tin Alloy (Material CF).

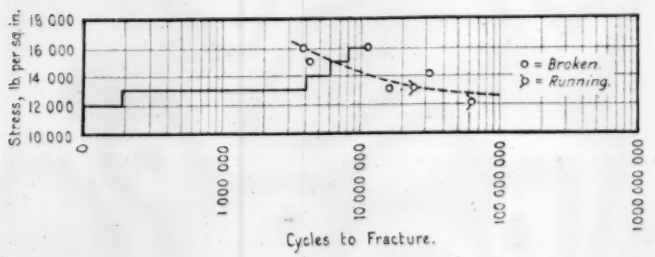

FIG. 7.—Curve Showing Rotating Cantilever Results on Copper, Cold Drawn (Material CL).

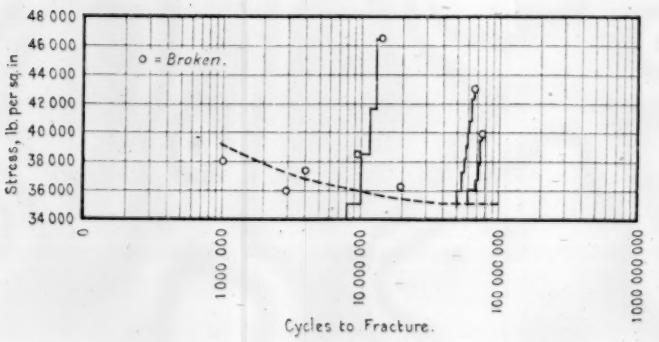

FIG. 8.—Curve Showing Rotating Cantilever Results on Aluminum Bronze (Material BJC).

Mr. Moore.

As the original paper of Mr. McAdam may not be readily available to the reader, the author has reproduced here in the accompanying Figs. 3 to 8 the graphs of the metals in question from reference (7) of the paper.[1]

Referring to Fig. 3, which represents McAdam's graph for annealed nickel, it is seen that the longest test run is approximately 325,000,000 cycles. At this point it is clear that the curve still continues to slope downward. It is difficult to understand how Mr. McAdam can interpret these tests as having established an endurance limit for nickel, especially when he has obtained a failure at 325,000,000 cycles. I see no evidence in these tests which would prevent one from assuming that the graph after passing the last plotted point continues to slope down. Even though the slope is somewhat more gradual at this point, the location of the endurance limit is clearly very much less definite than is found on steels or even as the writer has found with magnesium alloys. The notation in column 2 of Table VI, "300,000,000—no endurance limit reached" is obviously justified.

The same criticism applies to each of the Figs. 4 to 8. In Fig. 5 the author especially questions McAdam's liberty in flattening out the lower end of the graph after passing the last plotted point. In Figs. 6 and 8, Mr. McAdam gives undue weight in plotting the graphs to the last points which were not tests to failure at a specified stress, but were coaxing tests. As these latter points are the results of abnormal tests, there is considerable doubt as to the final shape of the graph. Fig. 7 most certainly has insufficient points from which to determine the shape of the graph. If the longest "running" test plotted here breaks within the next 40,000,000 reversals (which is about two-thirds of what it has already run) the graph will still slope down at a very rapid rate.

In quoting the values in column 2, Table VI, the author has used only the normal tests. The "coaxing" tests were omitted because their exact significance is not definitely known at this stage of progress in the study of fatigue of metals. Although Mr. McAdam has objected to some of the values given, the author feels that the objections have been satisfactorily answered and considers the values quoted as being entirely justified.

In reference to the fourth paragraph of Mr. McAdam's comments, it was not the intention of the author to misquote Mr. McAdam, but only to agree with him to a limited extent. Whereas Mr. McAdam is satisfied that *each* non-ferrous metal has a definite endurance limit,

[1] D. J. McAdam, Jr., "Endurance Properties of Alloys of Nickel and of Copper, Part I," *Transactions*, Am. Soc. Steel Treating, January, 1925.

Mr. Moore. the author would only go so far with him as to agree that *some* non-ferrous metals have a definite endurance limit. The author's contention that each non-ferrous metal has not as yet been proven to have a definite endurance limit is supported by the tests at the University of Illinois on monel metal, McAdam's own tests on nickel in particular, and the author's own tests on duralumin. In each of these cases no definite endurance limit was obtained.

Mr. McAdam has characterized the author's graphs for duralumin and those for monel metal at the University of Illinois as being abnormal. He states very definitely that "the normal rotating-cantilever stress-cycle graph for non-ferrous as well as for ferrous metals is a curve descending with gradually decreasing slope to a horizontal or nearly horizontal tangent." The author would like to call attention to a paper by Mr. McAdam given in *Chemical and Metallurgical Engineering* for December 14, 1921, in which he has plotted twenty-three graphs for various steels, every one of which is a straight line instead of a "curve descending with a gradually decreasing slope." Perhaps all these tests of Mr. McAdam were also abnormal. Upon careful examination of McAdam's own graphs it will be found also that in many cases of non-ferrous metals a straight line will represent the trend of the plotted points as fairly as a curve does. Mr. McAdam, in setting up his own work as a standard for others, has forgotten that the McCook Field and University of Illinois tests are run on a different type of machine (the rotating beam). In the cantilever machine the specimen is subjected to shearing stresses as well as bending stresses. In the rotating beam machine the specimen is subjected only to bending stresses. Might I suggest that this difference in the stresses produced may account for some of the discrepancy between the shapes of the graphs?

The author must take decided exception to Mr. McAdam's statement that "much more information will be obtained by running six specimens at 100,000,000 cycles than by running one specimen to 600,000,000 cycles." Most of the discussion on this paper seems to be centered around the fact that Mr. McAdam stops his tests too close to what he considers the point of tangency, before that point has been unquestionably determined. Referring to the author's Fig. 7, which contains the 600,000,000 cycle tests, there is no doubt that the graph has flattened out. These tests further demonstrate that there is no further change in the shape of the curve up to this point, which is very valuable information as this alloy has never been run before, and our present knowledge of fatigue of metals is at best very limited. Again it will be noticed that there are three of these 600,000,000 cycle

tests, two of which are on very similar alloys, and the third, although it has the same base element, uses a different added element. These combined tests, therefore, give confidence to the belief that the magnesium-aluminum and magnesium-copper alloys have a very definite endurance limit. **Mr. Moore.**

With the adoption of high-speed engines, rapidly vibrating apparatus, and accessories, the number of cycles of stress to which a part may be subjected during its useful life is very large. In the simple case of an automobile engine which has run 50,000 miles the crank shaft will have been subjected to 120,000,000 reversals of stress. The connecting rod of a steam engine after 10 years of normal service will have been subjected to almost 600,000,000 reversals. These are but very mild examples. It seems quite obvious that 100,000,000 cycles is a relatively small life for a test specimen which is to determine the practical working strength of a material for modern machine members.

These three tests at 600,000,000 are evidently many times more valuable than a dozen tests at 100,000,000.

MR. R. L. TEMPLIN[1] (*by letter*).—Both last year and the year before, the writer submitted test data relative to some rather long-time fatigue tests on heat-treated duralumin, as part of the discussion of previous tests reported by Mr. R. R. Moore.[2] At this time the writer is able to make a final report on these same tests. **Mr. Templin.**

Three specimens of 17 S-T rolled rod were tested as rotating beams under fiber stresses of 4000, 6000 and 8000 lb. per sq. in., respectively, and withstood the following number of stress cycles without failure:

SPECIMEN	STRESS, LB. PER SQ. IN.	NUMBER OF CYCLES
B	8 000	2 029 200 000
C	6 000	2 064 424 000
D	4 000	1 928 555 000

As a point of further interest, these specimens were tested full size in tension immediately after removal from the fatigue testing machines, in order to ascertain whether or not any marked changes had occurred in their static properties due to the exceptionally long-time fatigue tests. Below are tabulated the mechanical properties obtained from these static tests together with tests obtained previous to the starting of the endurance tests:

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[1] Chief Engineer of Tests, Aluminum Co. of America, New Kensington, Pa.

[2] Discussion of paper by R. R. Moore, "Resistance of Metals to Repeated Static and Impact Stresses," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 610 (1924); also discussion, *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 126 (1923).

Mr. Templin. Mechanical properties of the material before fatigue tests (tested September 15, 1921), were as follows: tensile strength 56,800 lb. per sq. in., proportional limit, 28,000 lb. per sq. in. and elongation in 2 in. (4D), 23.0 per cent.

Tests of specimens *B*, *C* and *D* after approximately 2,000,000,000 cycles at 8000, 6000 and 4000 lb. per sq. in., respectively (specimens tested July 15, 1925), gave the following results:

SPECIMEN	TENSILE STRENGTH, LB. PER SQ. IN.	PROPORTIONAL LIMIT, LB. PER SQ. IN.	ELONGATION IN GAGE LENGTH OF 4 DIAMETERS, PER CENT
B	59 220	28 000	21.5
C	58 370	28 000	22.9
D	58 010	24 000	22.1

NOTE.—The proportional limits were determined from stress-strain data obtained using a 2-in. Ewing extensometer which indicated total deformation to the nearest 0.00002 in.

The data just given would indicate that fatigue tests on the specimens has had practically no effect on the static properties of the material. The slight increase in tensile strength of the specimens may be accounted for by the aging of the specimens which normally occurs to a slight extent for a number of years.

SOFTENING OF HARD-ROLLED ELECTROLYTIC COPPER

By Norman B. Pilling[1] and George P. Halliwell[2]

Synopsis

A detailed study has been made, by means of tension tests, of the rates of softening of hard-rolled copper when reduced various degrees initially and heated at temperatures below 500° C. The quantitative effect of these variables is shown. The results indicate that copper has no minimum temperature of recrystallization but probably softens at a very slow rate even at atmospheric temperatures. Softening as shown by decrease in strength precedes visible recrystallization and has not been found to be initiated by a rise in hardness.

The effect of excessively long heating beyond that necessary to soften is shown to be slight at the lower temperatures, and the combination of amount of cold rolling, annealing temperature and length of anneal best suited for the production of highly ductile copper is discussed.

An annealing operation, which has for its purpose the removal of the hardness accruing from cold-working operations, may as a matter of convenience be divided into four periods:

1. Heating from atmospheric to the desired annealing temperature;
2. Softening at annealing temperature;
3. Continued or excess heating; and
4. Cooling from annealing to atmospheric temperature.

In practice these distinctive stages may insensibly blend together. It is the purpose of this paper to investigate these effects separately in copper of an average grade of purity, with the ultimate objective of formulating the conditions necessary for the production of copper having a high degree of ductility.

Although much of the treatment will consist in giving quantitative expression to facts previously sensed in a qualitative way, a somewhat new point of view will be developed regarding the practical stability of cold-worked metal and the existence of definite minimum recrystallization temperatures.

[1] The International Nickel Co., Bayonne, N. J.
[2] Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

MATERIAL OF TESTS

The copper used in this investigation was rolled from a single 225-lb. "BER" brand electrolytic wire bar, selected as representing a bar of average grade. With a few minor exceptions it was worked down completely in the Copper Mill of the Westinghouse Electric and Manufacturing Co., according to customary mill routine to specified final conditions under the supervision of Mr. C. R. Stevens. When hot-rolled to a 2-in. diamond section, a 10-lb. portion was cropped and millings were taken from the entire cross-section for chemical analysis, with the following result:

COMPOSITION OF WIRE BAR

Copper	99.94 per cent
Oxygen	0.066 "
Iron	0.003 "
Sulfur	0.002 "
Lead	nil
Tin	nil
Silicon	nil

A wire was drawn from a portion of the hot mill crop and its specific electrical resistance measured (Kelvin bridge) after annealing at 425° C. This was found to be 1.723 microhm cm. at 20.2° C., equivalent to a conductivity of 100.1 per cent.

Both chemical analysis and conductivity agree in indicating an average grade of copper.

ROLLING TREATMENT

The wire bar was hot rolled to a slab 3 in. wide, from which four lots of metal differing in degree of final reduction by cold rolling were obtained, as detailed in Table I. The rolling was done on a 12-in. rolling mill, and the finished sheets, after slitting to $\frac{3}{4}$-in. strip, were given one draw through a sizing die to round edged strap.

EXPERIMENTAL METHOD

The annealing was done in an electrically heated tube furnace in a protective atmosphere of purified nitrogen. The heating chamber consisted of a quartz tube projecting a considerable length from the furnace; it was fitted with gas seals and a manipulating arrangement whereby the load could be charged into the furnace or withdrawn to the cold end of the tube for cooling without entrance of air. In certain cases, a heating bath of oil or melted lead was substituted.

The test specimens on which tension tests were made were carefully machined, after annealing, to a reduced section $\frac{5}{8}$ in. wide by

2½ in. long. Practically all tests were made in triplicate on a Riehlé 5000-lb. testing machine with self-aligning grips designed for thin sheet metal.[1]

The measurement of ductility in thin strip encounters some difficulties not ordinarily of great moment in more robust sections. Reduction of sectional area at fracture is a useful criterion of ductility, and it has been shown that in a drastic edge bend of thin copper ribbon the extreme fiber may endure an unsupported strain almost equal to that of the necked-down portion of the tension test specimen just before fracture. It is therefore thought that a measurement derived from the maximum strain before fracture is of most direct utility in judging ability to permit severe unsupported tensile strain,

TABLE I.—COLD-ROLLED COPPER.

Lot	Preliminary Rolling			Final Rolling	
	Thickness		Grain Size before Final Cold Rolling, mm.	Thickness after Cold Rolling, in.	Reduction in Thickness by Cold Rolling, per cent
	After Hot Roll and Anneal, in.	After Cold Roll and Anneal, in.			
No. 4	0.267	0.104	0.018	0.0795[a]	23.8
No. 1	0.166	.....	0.017	0.0778[b]	53.2
No. 2	0.267	.....	0.020	0.0766[b]	71.2
No. 3	0.267	.....	0.020	0.0429[a]	84.0

[a] Not edged.
[b] A, C = side strips; B = central strip.

which may be considered one definition of ductility. The conventional reduction of area is not especially sensitive to changes in ductility in thin strip, not only on account of increased difficulties of measurement but to a peculiarity in the mechanics of fracture. The outer edges of the test specimen contract rapidly during the act of tearing and the extent of this contraction does not seem to be governed by the useful deformability of the metal, but is in fact greater in hard-rolled than in annealed copper. Careful measurements of reduction of area of small sections from enlarged photographs of fractures show little difference between these two conditions and conform to the observation of Jeffries.[2] This contraction at fracture, which may be a large fraction of the thickness of a thin strip, affects the width of a test specimen originally ⅝ in. wide to a relatively minor

[1] See Fig. 7, Tentative Methods of Tension Testing of Metallic Materials (E 8 – 24 T), *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part I, p. 858 (1925).

[2] *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 60, pp. 489, 490. Reduction of area of cold-drawn copper wire found to be 64.5 per cent; annealed copper wire, 76.0 per cent.

extent; for these reasons we have emphasized reduction of width at fracture as most truly indicative of plastic tensile deformability.

Consolidated test data, consisting of averages struck from each anneal, are given in Table II. The omission of reduction of width

TABLE II.—CONSOLIDATED TEST DATA.

(All results are average of 3 or more tests unless otherwise indicated)

Reduction by Cold Rolling, per cent	Annealing Temperature, deg. Cent.	Length of Anneal	Tensile Strength, lb. per sq. in.	Ductility		Grain Size, mm.
				Reduction of Width, per cent	Elongation in 2 in., per cent	
	...[a]	...	41 600	11.2	14.5	....
23.8	250	40 hours	38 100[c]	16.1[c]	26.2[c]	....
	400	1 "	33 300	26.8	52.6	0.023
	900	1 "	32 300	23.5	48.3	0.058
	...[a]	...	52 600	6.1	5.5	....
	100	1 hour	52 900	....	6.1	....
		74 days	52 900[b]	....	5.7[b]	....
		285 "	51 100[b]	....	8.5[b]	....
		359 "	51 800[b]	....	5.0[b]	....
	150	11 "	50 700	....	10.3	....
	175	12 hours	52 200	7.7	6.8	....
		25 "	51 000	7.5	8.1	....
	225	1 "	47 300	11.0	13.2	....
		4 "	41 200	15.3	22.5	....
		16 "	36 500	23.7	41.2	....
		64 "	35 400	27.5	47.0	0.013
	238	16 "	35 100	27.4	51.3	....
	250	0.5 "	39 800	16.4	30.8	....
		1 "	38 000	19.7	32.8	....
		4 "	35 000	26.9	48.1	....
		16 "	34 500	28.6	50.2	....
		64 "	34 400	28.1	51.1	0.013
	300	1.5 min.	52 700[b]	7.3[b]	7.5[b]	....
		3 "	48 700[b]	8.0[b]	9.5[b]	....
		6 "	37 400[b]	21.3[b]	33.5[b]	....
		10 "	35 700[b]	25.1[b]	45.7[b]	....
		30 "	34 600	28.8	52.7	....
		1 hours	35 500	....	38.8	....
		2 "	34 200	27.6	48.7	0.024
53.2		7 "	34 000	28.3	48.6	0.021
		24 "	33 800	28.6	50.6	0.028
	350	1 "	34 300	27.1	44.7	....
	400	2.6 sec.	42 700[b]	13.9[b]	20.0[b]	....
		5.2 "	34 600[b]	28.0[b]	48.0[b]	....
		9.9 "	34 500[b]	28.7[b]	54.5[b]	0.013
		19.9 "	34 500[b]	28.0[b]	51.2[b]	....
		0.5 hours	33 700	28.9	52.3	0.027
		1 "	33 900	29.1	45.0	0.020
		2 "	34 500	28.3	50.5	0.024
		7 "	33 400	28.4	51.3	0.026
		24 "	33 300	28.2	49.3	0.023
	500	1.1 sec.	49 000[b]	7.5[b]	6.5[b]	....
		1.6 "	38 000[b]	20.0[b]	30.5[b]	....
		2.4 "	34 700[b]	29.7[b]	54.2[b]	0.012
		5.4 "	34 600[b]	28.9[b]	51.7[b]	....
		0.5 hours	33 100	27.1	47.6	0.025
		1 "	33 300	27.8	46.8	....
		2 "	32 700	....	50.2	0.026
		7 "	32 800	28.2	49.9	0.024
		24 "	32 800	28.5	52.1	0.027
	600	1 "	32 800	27.6	48.5	0.023
	700	1 "	32 600	26.7	46.4	0.029
	800	1 "	32 300	24.9	43.5	0.037
	900	1 "	32 000	23.0	43.2	0.054
		24 "	32 000	17.8	40.1	0.126
	1000	1 "	31 800	20.0	43.5	0.102

[a] Not annealed.
[b] Single test.
[c] Two tests.

TABLE II (*Continued*).—CONSOLIDATED TEST DATA.

Reduction by Cold Rolling, per cent	Annealing Temperature, deg. Cent.	Length of Anneal	Tensile Strength lb. per sq. in.	Ductility		Grain Size, mm.
				Reduction of Width, per cent	Elongation in 2 in., per cent	
	...[a]	...	56 100	6.6	5.7	....
	100	1 hour	55 800	....	5.3	....
		74 days	55 500[b]	5.2[b]	7.2[b]	....
		285 "	54 200[b]	....	5.5[b]	....
		359 "	54 200[b]	....	6.5[b]	....
	150	11 "	49 700	....	15.8	....
	175	12 hours	52 600	8.6	10.4	....
		25 "	50 100	9.4	14.1	....
	200	1 "	52 300	8.3	9.0	....
	225	1 "	42 100	16.5	27.3	....
		4 "	37 500	21.5	40.6	....
		16 "	35 400	26.8	49.0	....
		64 "	35 600	28.3	50.5	0.016
	238	16 "	34 700	27.6	50.2	....
	250	0.5 "	37 500[c]	22.9[c]	42.5[c]	....
		1 "	35 600	24.9	44.7	....
		4 "	34 700	27.2	51.9	....
		16 "	34 300	28.5	52.9	....
		64 "	35 400	28.4	50.8	....
	300	1.5 min.	53 900[b]	....	6.2[b]	....
		3 "	47 200[b]	12.0[b]	15.2[b]	....
		6 "	35 100[b]	27.4[b]	50.5[b]	....
		10 "	35 300[b]	26.5[b]	51.5[b]	....
		30 "	35 300	28.2	50.6	....
		1 hours	35 100	27.3	47.0	....
		2 "	35 100	28.5	50.9	0.019
71.2		7 "	35 000	28.7	53.6	0.020
		24 "	35 100	28.6	54.3	0.020
	350	1 "	34 300	26.5	45.0	....
	400	2.6 sec.	37 000[b]	24.0[b]	36.2[b]	....
		5.1 "	35 400[b]	....	51.7[b]	....
		10.5 "	34 800[b]	29.2[b]	52.0[b]	0.012
		20.1 "	35 200[b]	28.4[b]	50.7[b]	....
		0.5 hours	34 700	27.7	50.5	0.022
		1 "	34 200	26.5	46.6	....
		2 "	35 000	27.8	47.3	0.023
		7 "	34 400	28.4	51.5	0.020
		24 "	34 300	28.4	51.1	0.024
	500	1.2 sec.	55 600[b]	6.8[b]	6.5[b]	....
		1.9 "	35 900[b]	28.2[b]	48.2[b]	0.011
		2.4 "	35 400[b]	28.5[b]	50.0[b]	....
		5.3 "	34 900[b]	29.3[b]	52.7[b]	....
		0.5 hours	34 100	26.7	50.5	0.024
		1 "	33 100	26.2	48.0	....
		2 "	33 700	26.8	51.8	0.025
		7 "	33 500	25.3	47.1	0.025
		24 "	33 600	26.7	52.7	0.024
	600	1 "	32 500	25.6	46.2	0.025
	700	1 "	32 400	24.2	42.9	0.033
	800	1 "	32 400	22.8	43.3	0.041
	900	1 "	32 900	19.1	39.0	0.067
		24 "	33 100	15.4	37.6	0.136
	1000	1 "	32 700	17.7	38.1	0.083
	...[a]	...	61 600	....	5.6	....
	225	6 min.	47 100[c]	10.4[c]	14.0[c]	....
84.0		12 "	41 800[c]	15.6[c]	29.2[c]	....
	400	1 hour	34 700	22.7	47.4	0.017
	900	1 "	29 500	17.9	35.9	0.056

[a] Not annealed.
[b] Single test.
[c] Two tests.

in certain tests indicates the occurrence of an oblique fracture, the attempted measurement of which would yield an equivocal result.

The tensile properties of the cold-rolled metal which formed the starting point of the investigation are correlated in Fig. 1. The

number of points is not great enough to define closely the initial effects of cold rolling in raising the strength and lowering the ductility, but they are suggestive of a somewhat more rapid rate of increase in the initial stages than previously found by Mathewson and Thalheimer.[1]

The range of cold reductions covered is not extreme but includes the commercial range. Under certain as yet undefined conditions of rolling, excessive reduction may result in softening rather than

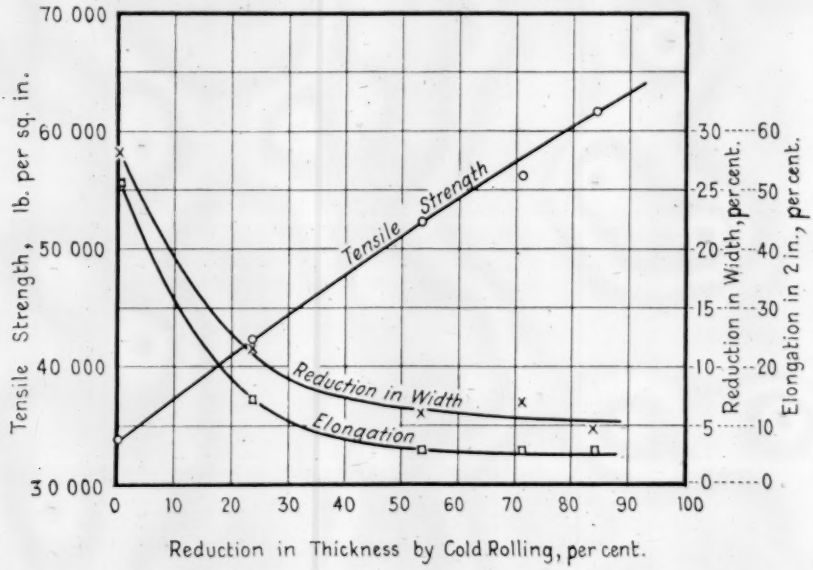


FIG. 1.—Effect of Cold Rolling on Tensile Properties of Electrolytic Copper.

continued hardening, as reported by Rawdon and Mutchler;[2] but it is significant to note that Fogler and Quinn[3] failed to find this effect when the temperature of the rolled sheet was prevented from rising. The maximum reduction in the present experiments was below this doubtful range.

EFFECT OF HEATING TO AND COOLING FROM THE ANNEALING TEMPERATURE

The rate of heating to the annealing temperature preceding softening is without appreciable influence on the result. The grain

[1] C. H. Mathewson and E. M. Thalheimer, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 55, p. 467 (1916).

[2] H. S. Rawdon and W. H. Mutchler, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 70, p. 342 (1924).

[3] M. F. Fogler and E. J. Quinn, *Transactions*, Am. Inst. Mining and Metallurgical Engrs. (1925)

size of a specimen heated quickly by immersion in molten lead did not consistently differ from a similar specimen heated slowly by radiation within a furnace.

The rate of cooling from the annealing temperature is without effect on either strength or ductility, as the following tests comparing slow cooling with quenching show (Table III). A group of six specimens previously rolled 53 per cent were annealed one hour at 400° C. in nitrogen, a treatment normally resulting in high ductility; three of these were removed and quenched in cold water, the other three

TABLE III.—EFFECT OF QUENCHING ANNEALED ELECTROLYTIC COPPER.

REDUCTION IN COLD ROLLING, 53 PER CENT.

Annealing Temperature, deg. Cent.	Cooling Method	Tensile Strength, lb. per sq. in.	Reduction of Width, per cent	Elongation in 2 in., per cent
400	Quench	34 200	27.6	52.7
		33 800	28.1	53.2
		33 900	27.5	53.2
		Average 34 000	27.7	53.0
	Slow	33 700	....	52.2
		33 900	....	51.5
		33 900	27.8	52.2
		Average 33 800	27.8	52.0
900	Quench	32 600	18.4	45.5
		32 100	21.7	48.0
		32 200	22.6	50.0
		Average 32 300	20.9	47.8
	Slow	32 300	23.6	46.2
		32 400	22.0	50.5
		32 700	21.1	46.0
		Average 32 500	22.2	47.6

cooling slowly in one end of the furnace tube. Another group was similarly treated following a one-hour anneal at 900° C., the normal result of which is a coarse-grain metal of poor ductility.

SOFTENING OF COLD-ROLLED COPPER

It is evident from the literature[1] that the range of annealing temperatures yielding greatest ductility lies below 600° C. This has been confirmed, and for this reason principal attention will be given to softening in the temperature range 100 to 500° C. At temperatures higher than 600° C. even brief annealing periods tend toward coarsening of the grain and inferior ductility.

The separation of the period of softening from that of excess heating requires the determination of softening rates. For this pur-

[1] C. H. Mathewson and E. M. Thalheimer, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 55, p. 470 (1916); G. V. Cæsar and C. G. Gerner, *Transactions*, Am. Inst. Metals (1916); E. S. Bardwell, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 49, p. 758 (1914).

pose, annealing experiments in which the length of heating was controlled were made at carefully regulated temperatures between 99 and 500° C. in liquid heating baths. The anneals at 99° C. were made in a light oil bath heated by a water vapor thermostat. The temperature of this being fixed by the barometric pressure, the temperature deviation was probably less than 0.5° C. even in anneals lasting many months. Above 100° C. up to 300° C., an electrically heated, agitated bath of heavy tempering oil was used; at 400 and 500° C. melted lead was the heating medium. The advantage of a liquid heating medium lies in its ability to transfer heat quickly to the test specimen, minimizing the uncertainty of the preheating period, and in the ease with which the temperature can be maintained constant. Heating was terminated by quenching in water.

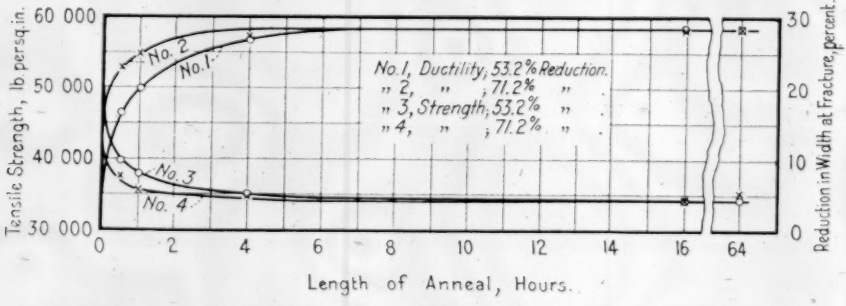


FIG. 2.—Effect on Physical Properties of Annealing for Various Lengths of Time.

Typical features of the softening process may be noted in Fig. 2, which shows the decrease in tensile strength and increase in ductility induced by heating at 250° C. The rate of softening, as indicated by the decrease in strength, gradually diminishes, and the final approach to complete softening occurs very slowly. Recovery of ductility lags behind softening. After four hours' heating at 250° C. the reduction of width has reached 95 per cent of its possible development; the microscope shows no trace of deformed crystallin material. At 16 hours the change is definitely complete and additional heating up to 64 hours resulted in no further change of structure.

The actual relation between softening and length of heating depends on the temperature of annealing and on the amount to which the metal has previously been cold worked. It can be demonstrated, however, that the softening curves exemplified in Fig. 2 have a common form irrespective of these variables. A mathematical analysis of these curves could be made and the various parameters determined,

but it is thought that this fact can be clearly demonstrated in another and more generally useful way.

A sufficient measure of the rate of softening of a cold-worked metal, assuming that all its softening curves were identical in form, would be the length of time required for complete softening. The measurement of this quantity by inspection of curves such as are shown in Fig. 2 would involve determination of the point of tangency to the horizontal coordinate, which would require a higher power of discrimination than the data afford. It has been observed, however

TABLE IV.—SOFTENING CHARACTERISTICS OF COLD-ROLLED COPPER.

Initial Cold Rolling, per cent	Annealing Temperature, deg. Cent.	Tensile Strength, lb. per sq. in.		Softening Time		Annealing Time		Softening	
		Cold Rolled	Fully Annealed[1]	70 Per Cent Softening	Complete Softening[2]	Duration	Per Cent	Measured Tensile Strength, lb. per sq. in.	Per Cent
53.2	225	52 600	34 700	5.2 hr.	104 hr.	1 hr.	1.0	47 280	29.6
						4 "	3.9	41 230	64
						16 "	15	36 530	90
						64 "	62	35 360	96
	250	52 600	34 400	30 min.	10 hr.	0.5 "	5	39 760	70
						1 "	10	38 000	80
						4 "	25	35 030	97
	300[a]	52 600	34 000	2.0 min.	40 min.	3 min.	0.8	48 700	21
						6 "	8.2	37 400	82
						10 "	18	35 700	91
						30 "	68	34 600	92
71.2	225	56 100	35 600	68 min.	23 hr.	1 hr.	4.3	42 100	68
						4 "	17	37 460	91
						16 "	70	35 360	100
	250	56 100	35 400	8.1 min.	2.7 hr.	0.5 "	18.5	37 500	90
						1 "	37	35 600	99
	300[a]	56 100	35 000	37 sec.	12.4 min.	3 min.	2.4	47 200	42
						6 "	27	35 100	99

[1] From Fig. 7.
[2] Time to soften 70 per cent × 20.
[a] Time required to heat in oil bath at 300° C. = 2.7 min.

that the time required to soften a definite fraction, say 70 per cent, of the total can be readily determined and the relative rates of softening under different conditions thereby be compared. A comparison of the strengths of partially annealed copper, in which the annealing temperature varied from 225 to 500° C., indicated that the ratio of the time required for complete softening to the time required for 70-per-cent softening was fairly definite, although the actual periods of time involved ranged from hours to seconds. This ratio was between 10 to 1 and 20 to 1, and there seemed to be no definite connection between it and annealing temperature or previous cold reduction.

It is believed that the latter figure is not far from the truth and this has been taken as a conservative multiplying factor.

Now if softening and annealing time are expressed in purely relative terms, such as percentage softening and percentage time required for complete softening, the curves for all annealing temperatures and degrees of cold reduction become identical. The data derived in Table IV are shown graphically in Fig. 3. This curve, which might be called the basic softening curve of copper, represents the progressive softening of hard-rolled copper when heated at any

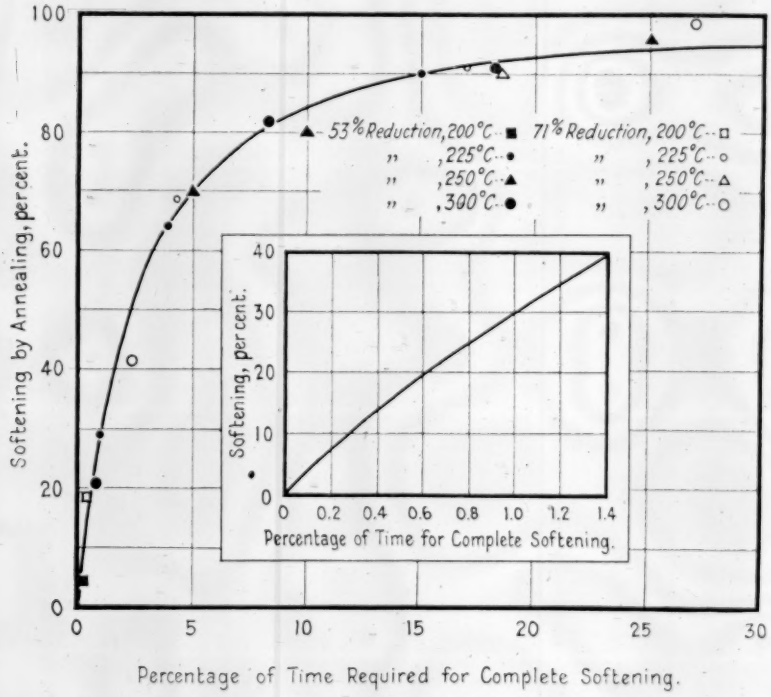


FIG. 3.—Basic Softening Curve for Cold Rolled Electrolytic Copper.

fixed temperature. It shows that within the limits of temperature and cold rolling included, both affect only the rate at which the change from the work-hardened to the annealed state progresses. The smoothness of this curve suggests an essential continuity in this change, even in the early period of sub-microscopic alteration. It is interesting to note the extremely rapid rate at which softening begins; thus 10 per cent softening occurs in about 0.3 per cent of the total time required to complete it, and 80 per cent occurs within the first 7 per cent of time.

The microstructural features of recrystallization have received much attention from previous investigators, and the general facts are well known.[1] We wish to emphasize the point that the change from the cold-worked to the soft condition proceeds in advance of the development of a visible recrystallization. To demonstrate this, a series of micrographs has been assembled (Fig. 11) showing the early stages of softening in copper initially reduced 53 per cent by cold rolling. These have been selected from comparatively brief anneals at various temperatures which yielded progressively increasing amounts of softening. In comparison with the original structure it will be seen that no recrystallization, in the sense of new grains microscopically detectible under high power, appears until a softening of about 30 per cent has occurred. The initial change corresponding to this softening is indicated by a partial recovery of etching contrast between the original strain-hardened grains (Figs. 11 (*a*) and (*b*)) which still retain their distorted shape. It is thought that this indicates that they have undergone a really essential change. With further softening, the recrystallized grains, chiefly segregated in particular areas, increase in amount, but not in proportion to the total softening. A softening of 69 per cent (Fig. 11 (*e*)) corresponds to a structure containing by a conservative estimate not more than 25 per cent of recrystallized grains. Visible recrystallization is therefore a comparatively late stage in the reorganization of the unstable strain-altered structure.

The increase in strength and hardness which seems to be well established as an initial stage in the softening of brass[2] does not occur in the case of electrolytic copper. Although this effect has been specifically looked for, the authors have invariably found that the first result of heating is to lower the strength and hardness (Rockwell), notwithstanding the recently announced result of Angus and Summers[3] to the contrary.

EFFECT OF TEMPERATURE ON RATE OF SOFTENING

From the basic softening curve it is possible to estimate the length of time required to complete softening under a given set of conditions provided the percentage softening resulting from a heating period of measured duration is known. This has made it feasible to assign a quantitative value to the softening rate at temperatures

[1] See for example, Jeffries and Archer, "The Science of Metals," p. 85 (1924).

[2] C. H. Mathewson and A. Phillips, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 54, p. 613 (1916); W. H. Bassett and C. H. Davis, *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 60, p. 428 (1919).

[3] *Journal*, Inst. Metals, preprint (1925).

much below those of rapid change. In Table V, the rates of softening at temperatures between 100 and 300° C. of metal having had an initial cold reduction of from 24 to 84 per cent are derived. The effect of softening temperature on softening rate is shown in Fig. 4 for two degrees of initial cold reduction, namely, 53 and 71 per cent. The rapid change in rate with temperature necessitates a logarithmic plot, and for convenience the vertical time scale is given on several separate scales as seconds, minutes, etc., as the softening period changes from one order of magnitude to another.

TABLE V.—RATE OF SOFTENING OF COLD-ROLLED COPPER.

Annealing Temperature, deg. Cent.	Initial Reduction, per cent	Tensile Strength, lb. per sq. in.			Softening, per cent	Annealing Time		Time Required for Complete Softening
		Cold Rolled	Fully Annealed[a]	Observed		Duration	Per Cent[b]	
100	53.2	52 600	36 400	51 120	9.1	285 days	0.25	310 yrs.
				51 800	4.9	359 "	0.11	900 "
	71.2	56 100	37 300	55 500	3.2	74 "	0.076	270 "
				53 740	12.6	285 "	0.35	223 "
				53 830	12.1	359 "	0.34	290 "
150	53.2	52 600	35 600	50 600	11.8	11 days	0.33	9.1 yrs.
	71.2	56 100	36 600	49 600	33.3	11 "	1.12	2.7 "
175	53.2	52 600	35 300	51 000	9.4	25 hr.	0.24	430 days
	71.2	56 100	36 400	52 600	17.8	12 "	0.52	96 "
				50 100	30.6	25 "	1.15	91 "
200	53.2	52 600	35 000	51 830	4.5	1 hr.	0.12	33 days
	71.2	56 100	35 900	52 270	19	1 "	0.40	10.4 "
225	53.2	52 600	34 700	40 100	70	5.2 hr.	5	104 hr.
	71.2	56 100	35 600	41 700	70	68 min.	5	23 "
	84.0	61 600	35 700	47 200	55.6	6 "	2.8	3.6 "
				41 900	75.1	12 "	6.3	3.2 "
250	23.8	41 600	34 100	38 100	46.6	40 hr.	2.0	2000 hr.
	53.2	52 600	34 400	39 900	70	30 min.	5	10 "
	71.2	56 100	35 400	41 600	70	8.1 "	5	2.7 "
300	53.2	52 600	34 000	39 600	70	2 min.	5	40 min.
	71.2	56 100	35 000	41 300	70	37 sec.	5	12 "

[a] From Fig. 7.
[b] From Fig. 3.

There are several features of softening revealed calling for special note: (1) The metal reduced 71 per cent softens consistently faster than the 53 per cent by a ratio of 4 or 5 to 1. (2) Softening rate increases extremely rapidly as temperature rises; between 200 and 300° C. an increase of 15° C. about doubles the softening rate. At 200° C. complete annealing is a matter of many days; at 300° C. of as many minutes; and at 400° C. seconds. (3) Heating at temperatures as low as 100° C. for a period of many months develops a softening in quantitative agreement with the results at higher temperatures (Figs. 12 (*b*) and (*c*)). (4) With the method of plotting adopted, the softening rate – temperature curve becomes very nearly a straight

line at temperatures below 225° C. Nothing in the shape of this curve suggests its termination at a particular temperature, but its continuous nature makes it reasonably certain that even at atmospheric temperatures the unstable work-hardened state is slowly but continuously reverting to the stable soft state. There is, properly speaking, no definite temperature at which copper acquires the ability to soften,

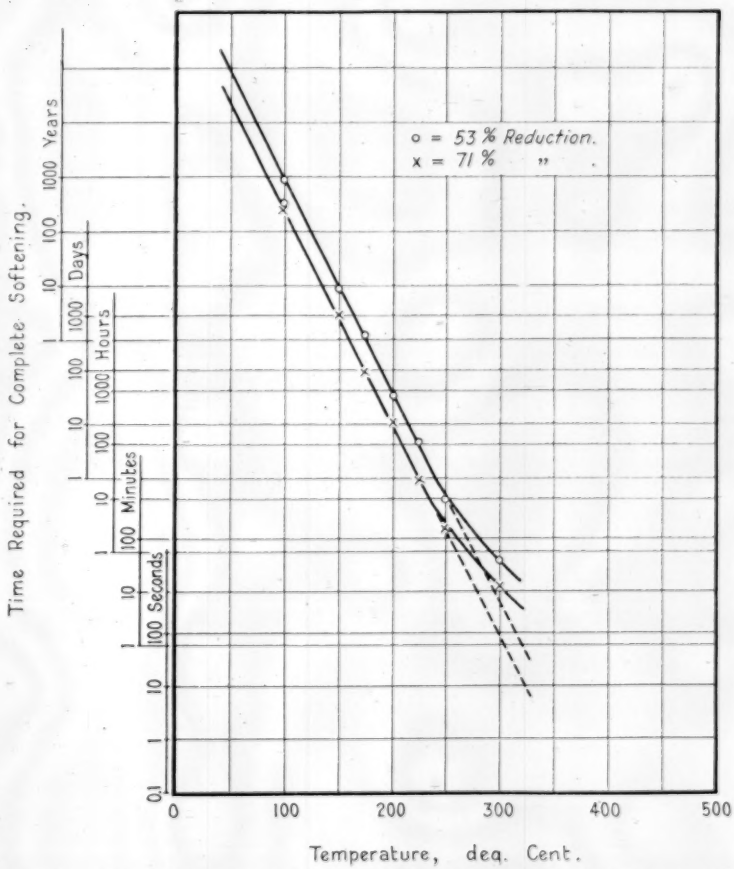


FIG. 4.—Effect of Temperature on Softening Rate of Hard Rolled Copper.

but rather the rate of softening becomes so small as ordinarily to escape observation. From Figs. 3 and 4, an initial cold reduction of 71 per cent would require 50 years at atmospheric temperature to show a softening of 1 per cent.

Above 400° C. the time required to soften becomes almost incredibly small, and for even a very light weight of metal the time required to

bring it to temperature may be many times more than sufficient for its complete softening. Although attempts were made to measure annealing rates at 400 and 500° C., the second or two required to heat a single 2-oz. test specimen to temperature after plunging in a lead bath was too great a fraction of the total annealing time to make results other than suggestive. In a rough general way these very brief anneals check the order of magnitude of the softening period

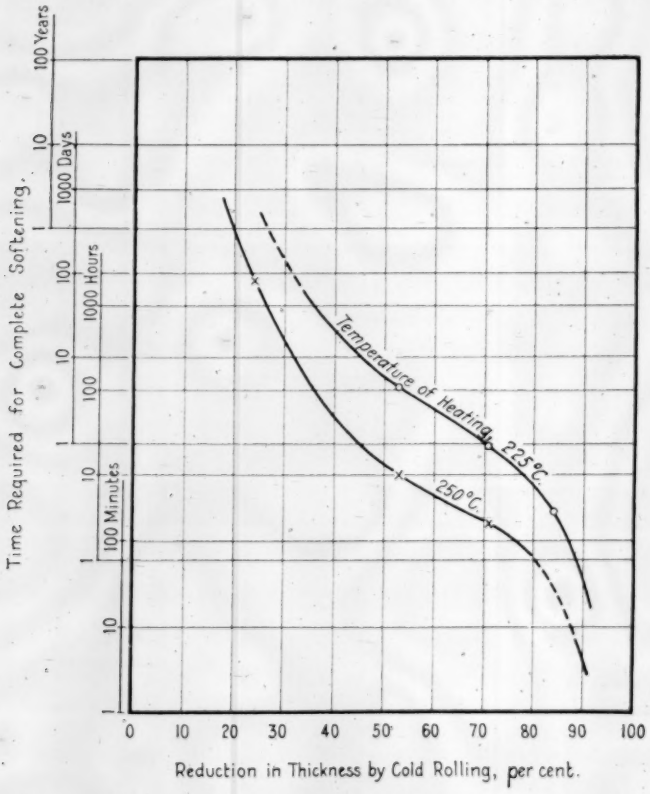


FIG. 5.—Effect of Cold Rolling on Annealing Rate of Copper.

indicated by the dotted extrapolation of the curves of Fig. 4. The following anneals produced completely softened test specimens of high ductility; the time indicated was from immersion in lead to quenching in water:

REDUCTION, PER CENT	TEMPERATURE, DEG. CENT.	TIME, SECONDS
53	400	5.2
	500	2.4
71	400	5.1
	500	1.9

EFFECT OF REDUCTION BY COLD ROLLING ON SOFTENING RATE

The effect of degree of cold rolling on rate of softening at two fixed temperatures, namely, 225 and 250° C., is shown in Fig. 5. It is quite evident that softening rate is enormously affected by the degree of cold rolling; for example, the relative rates following 84 and 24-per-cent reductions, respectively, are as 5000 to 1. The two curves show a double flexure and approach zero reduction and 100 per cent reduction assymptotically. This shows the greatly increased apparent stability of copper which has received but a moderate amount of cold rolling, and is suggestive at the other extreme of the difficulty of preserving the full effects of high degrees of cold rolling, especially if the temperature of the metal is allowed to rise.

TABLE VI.—EFFECT OF LENGTH OF ANNEAL.

TEMPERATURE, 400° C. REDUCTION, 53 PER CENT

ANNEALING TIME	REDUCTION OF WIDTH, PER CENT	TENSILE STRENGTH, LB. PER SQ. IN.
10 sec	28.7	34 500
30 min	28.4	33 700
24 hr	28.2	33 300

EFFECT OF PROLONGED HEATING ON STRENGTH AND DUCTILITY

From the extreme rapidity with which softening occurs at the higher temperatures, it is obvious that commercial annealing must consist almost entirely of excessively long heating, considered from the viewpoint of completion of the physical actions involved. This aspect of annealing has been examined by comparing the strength and ductility of copper which has been annealed just to completion with the same properties resulting from holding at temperature for many hours.

At 400° C. the effect shown in Table VI has been noted. Extending the heating period more than 8000 fold beyond the softening period has resulted in changes in strength and ductility which, although small, are believed to be real. The tendency is for a slight deterioration in ductility and loss of strength accompanied by an increase in grain size (Fig. 13). The comparatively ineffective result of prolonged heating up to 24 hours has been confirmed at 300 and 500° C. for both 53 and 71 per cent reductions (Fig. 6).

Whereas this effect is small enough at temperatures below 500° C. to be of little practical concern, the tendency indicated becomes increasingly apparent as the heating temperature is raised until it

becomes a dominant action at temperatures such as 900° C., as indicated in Table VII. This marked deterioration is accompanied by an equally marked increase in grain size.

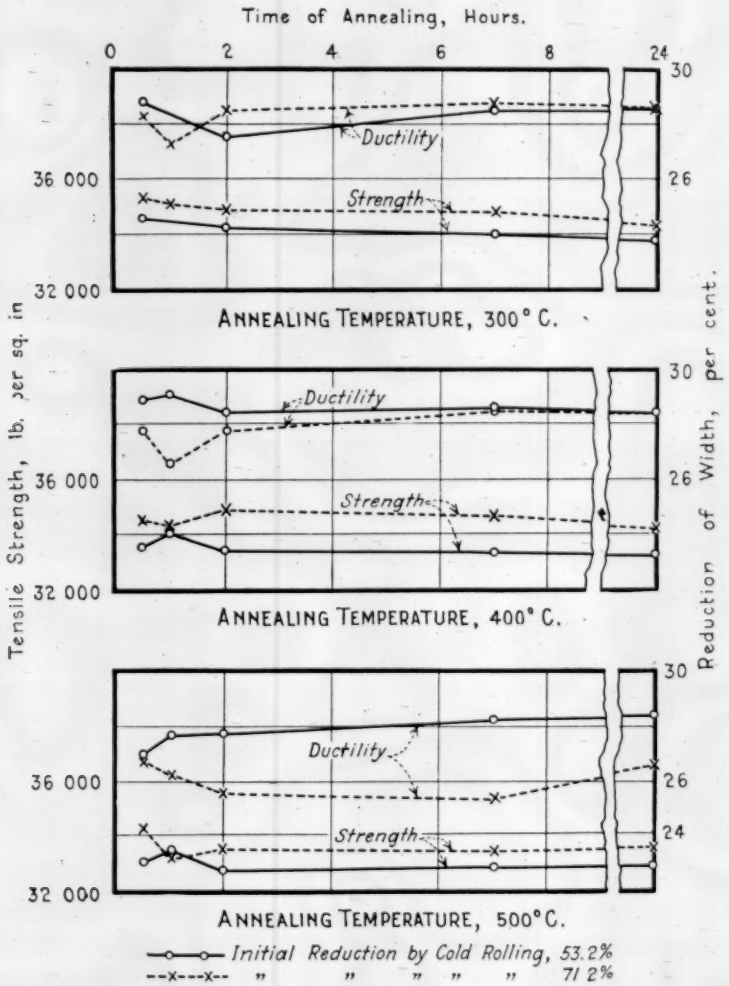


FIG. 6.—Effect of Prolonged Annealing on Ductility and Strength of Electrolytic Copper.

The essential and fortunate point is the extreme slowness with which this deterioration proceeds as compared with softening. It appears that roughly 10,000 times the softening period is required before the loss in ductility becomes great enough to be certainly observable in the tension test. Judging from the slope of the curves

in Fig. 4, softening is so rapid at 900° C. that even this multiple is exceeded many fold within one hour.

EFFECT OF ANNEALING TEMPERATURE ON STRENGTH AND DUCTILITY

The general effects of annealing temperature on strength and ductility have been described by Mathewson and Thalheimer,[1] Cæsar and Gerner[2] and others. These investigators have shown the decrease

TABLE VII.—EFFECT OF PROLONGED HEATING AT 900° C.

REDUCTION PER CENT	ANNEALING TIME, HOURS	REDUCTION OF WIDTH, PER CENT	TENSILE STRENGTH, LB. PER SQ. IN.	GRAIN SIZE, MM.
53	1	23.0	32 000	0.054
	24	17.8	31 000	0.126
71	1	19.6	32 600	0.067
	24	15.4	33 200	0.136

in ductility following annealing at the higher temperatures, but have not attempted to distinguish between the properties of partially and fully annealed metal. Fig. 7 shows the results of annealing to completion or beyond. In conformity with the view that there is no

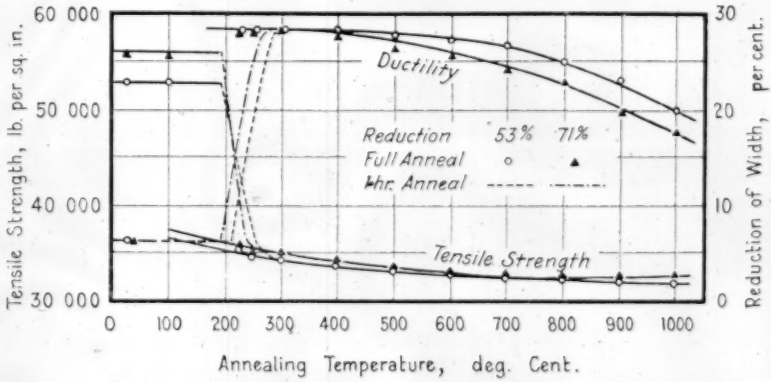


FIG. 7.—Effect of Annealing Temperature on Ductility and Strength of Electrolytic Copper.

definite initial temperature of softening, these curves are drawn as two unconnected branches. For the sake of adding perspective, curves representing the effect of one hour's heating at the lower temperatures are dotted in. Highest ductility tends to be obtained at the lowest annealing temperature. Up to 400° C., the two reduc-

[1] *Transactions*, Am. Inst. Mining and Metallurgical Engrs., Vol. 55, p. 469 (1916).
[2] *Transactions*, Am. Inst. Metals (1916).

tions yield practically equal ductility, and the decrease due to increased annealing temperature is very slight. At 400° C. the curve representing 71 per cent reduction begins to drop, and from there on ductility decreases with rising annealing temperature; at 1000° C. the ductility

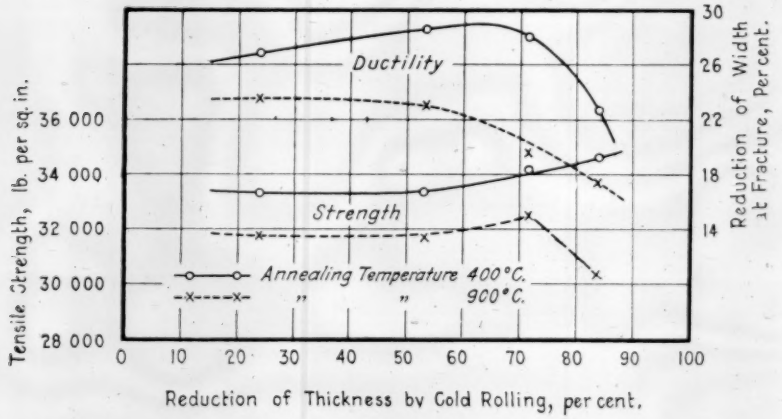


FIG. 8.—Effect of Cold Rolling Before Annealing on Ductility and Tensile Strength of Electrolytic Copper.

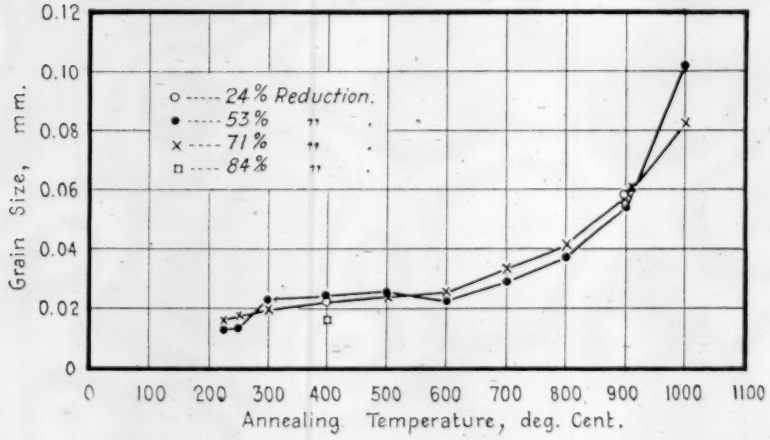


FIG. 9.—Effect of Annealing Temperature on Grain Size of Electrolytic Copper.

is but 60 per cent of its possible development. An initial reduction of 53 per cent results in less loss of ductility at the higher annealing temperatures. It follows, therefore, that the effects of different degrees of rolling do not vanish on annealing.

There does not appear to be a very definite connection between the tensile strength and ductility of fully annealed copper. The decrease in strength following annealing at progressively higher temperatures below 500° C. is reflected in very little change in ductility, and the great loss in ductility following high temperature annealing occurs with little further change in strength. Nevertheless, the generalization is true, that the strongest fully annealed copper is most ductile.

Fig. 14 illustrates the microstructural changes of 71 per cent reduced metal annealed at progressively higher temperatures.

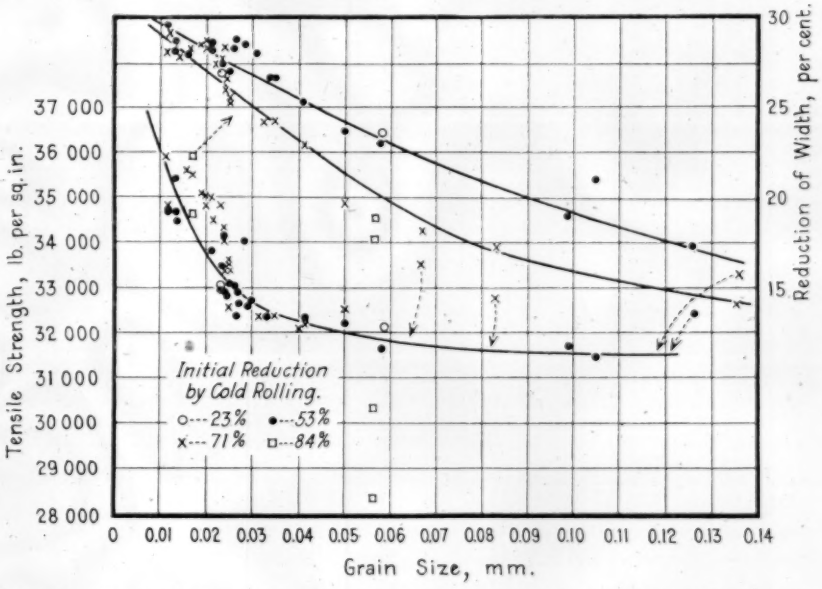
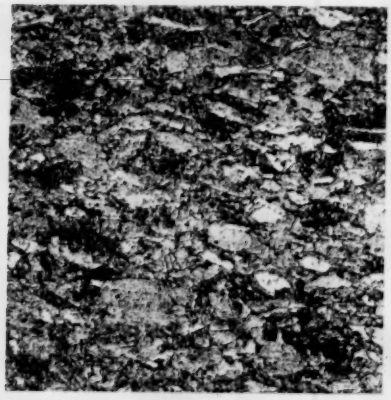


FIG. 10.—Effect of Grain Size on Ductility and Tensile Strength of Electrolytic Copper.

EFFECT OF COLD ROLLING PRECEDING ANNEALING ON DUCTILITY AND STRENGTH SUBSEQUENTLY DEVELOPED

The dependence of the tensile properties on the amount of prior cold rolling is shown in greater detail in Fig. 8, in which ductility and strength are shown following: (1) Annealing in the best temperature range for production of high ductility; and (2) annealing in the high temperature range yielding inferior ductility. The temperatures chosen for this purpose were 400 and 900° C. Following the 400° C. anneal, ductility rises slowly through a flat maximum as the amount of cold rolling increases, declining rapidly beyond 70-per-cent reduction.



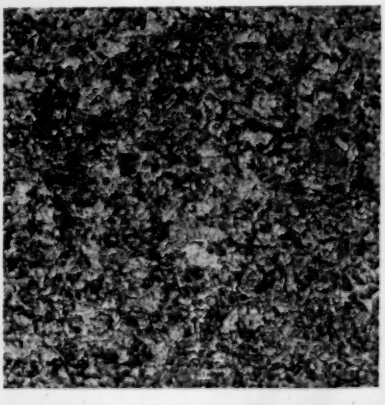
(*a*) 12 per cent softening, 150° C., 11 days.

(*b*) 21 per cent softening, 300° C., 3 minutes.

(*c*) 30 per cent softening, 225° C., 1 hour.

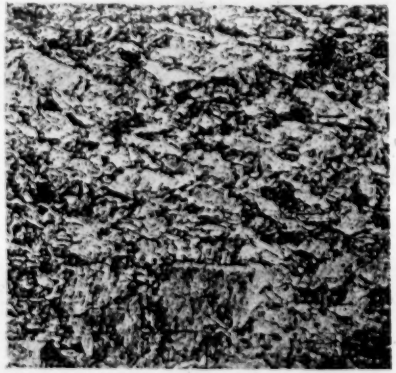
(*d*) 52 per cent softening, 400° C., 3 seconds.

(*e*) 69 per cent softening, 250° C., ½ hour.



(*f*) 90 per cent softening, 225° C., 16 hours.

FIG. 11.—Softening of Copper Reduced 53 per cent (× 100).

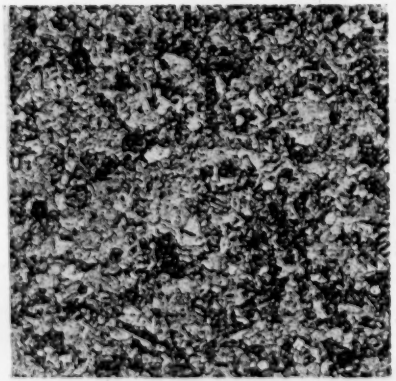


(*a*) Cold rolled 53 per cent. Not annealed.

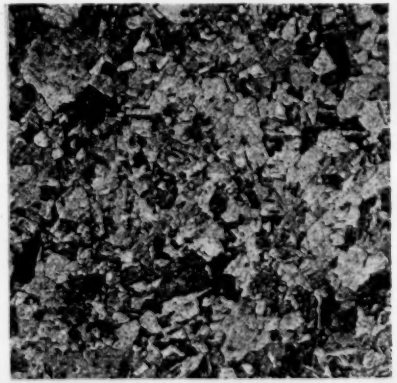
(*b*) Initial reduction 71 per cent. Heated at 99° C., 359 days.

(*c*) Initial reduction 53 per cent. Heated at 175° C., 25 hours.

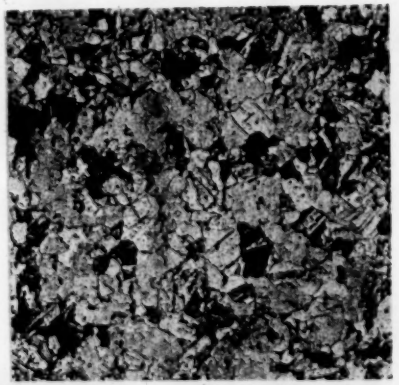
FIG. 12.—Effect of Low Temperature Heating (× 100).



(*a*) 10 seconds.

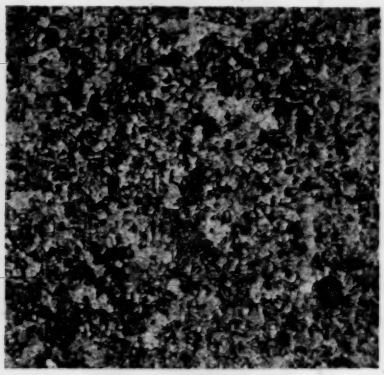


(*b*) 30 minutes.

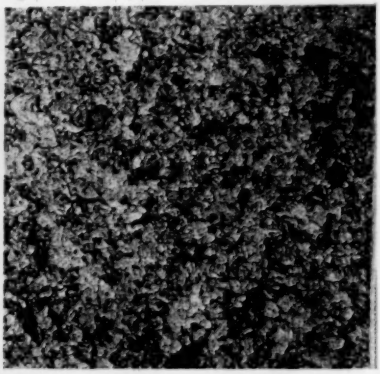


(*c*) 24 hours.

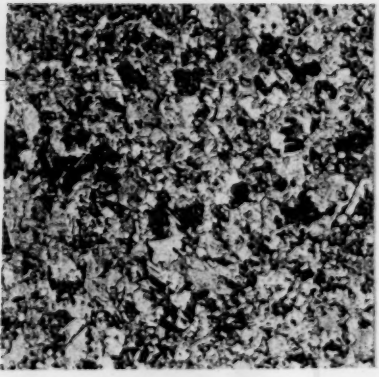
FIG. 13.—Effect of Length of Heating at 400° C.; Initial Reduction 53 per cent (× 100).



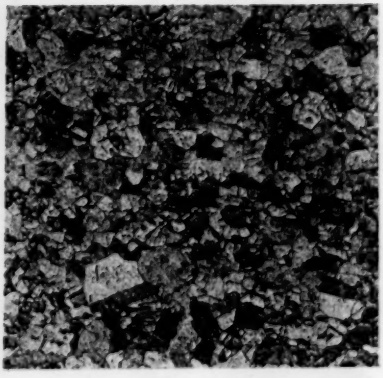
(*a*) 225° C., 64 hours.



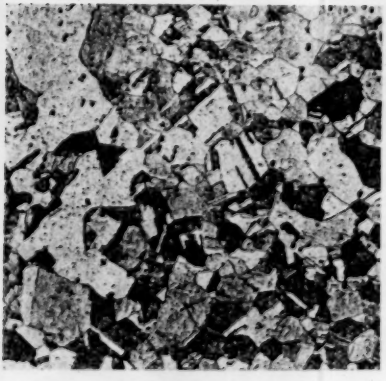
(*b*) 250° C., 64 hours.



(*c*) 300° C., 7 hours.



(*d*) 500° C., 7 hours.



(*e*) 700° C., 1 hour.

(*f*) 1000° C., 1 hour.

FIG. 14.—Fully Annealed Copper, Initial Reduction 71 per cent (× 100).

After annealing at 900° C. the result is similar, the decline in ductility beginning at 60 per cent. In both cases strength increases with reduction, but the highest reduction preceding a 900° C. anneal shows a discontinuous drop in strength.

It appears that there is a wide range of cold rolling (30 to 70-per-cent reduction) in which high ductility may subsequently be developed by proper annealing; if this range is exceeded, ductility is adversely affected. The best combination is about 65-per-cent reduction followed by annealing at 400 to 500° C.; the worst is an excessively great reduction followed by a very high temperature anneal.

EFFECT OF GRAIN SIZE ON DUCTILITY AND STRENGTH

The coincidence of a rapid increase in grain size in the temperature range in which ductility decreases rapidly (Fig. 9) suggests a possibly close relation between the two. In general, the relation is definite (Fig. 10), ductility diminishing as grain size increases, yet it will be noted that the effect of initial cold rolling persists throughout. As a result, copper of a given grain size has poorer ductility the greater the initial cold rolling of the metal from which it was derived. This seriously limits the usefulness of the microscope in estimating ductility from microstructure. It should be noted that the loss of ductility produced by excessively long heating at high annealing temperatures is shown by these curves to be merely a coarse grain effect, and the evident superiority of copper which has not greatly exceeded the time for complete softening is likewise a result of the extremely fine grain which it has. On the other hand, coarse grains frequently develop at annealing temperatures of 400 to 500° C. which have no special effect on the ductility, perhaps due to their great intricacy of shape.

As shown in Fig. 10, strength increases with increasing fineness of grain, yet coarse-grained copper, corresponding to an annealing temperature of 800° C. and over, is subject to great fluctuations in strength.

A NOTE ON THE MICROSTRUCTURE OF ALUMINUM-IRON ALLOYS OF HIGH PURITY

By E. H. Dix, Jr.[1]

Synopsis

In a previous paper the author has reported the results of a metallographic study of the occurrence of iron and silicon in aluminum of ordinary purity. The present paper deals with the occurrence of iron in alloys prepared from aluminum of much greater purity than was previously available. The microstructures of slowly cooled and chill-cast specimens are illustrated as well as some striking examples of segregation obtained in a small chill-cast specimen ¾ in. in diameter. It is further shown how this segregation was eliminated. A marked difference in the apparent eutectic concentration, as indicated by chill-cast and slowly cooled structures, is pointed out. The chill-cast eutectic structure is exceedingly fine and has, therefore, been illustrated at magnifications as high as 5000 diameters. The effect of annealing for one week at a temperature close to the eutectic temperature is described. The effects on the pure iron-aluminum constituent of the etching reagents commonly employed in the metallography of aluminum alloys are given.

The study of the structure of aluminum alloys has been made extremely difficult by the presence of appreciable amounts of iron and silicon in all commercial aluminum. Material of much higher purity has recently become available for such study through the development by the Aluminum Co. of America of a new process for electrolytically refining aluminum.[2] Thus, it is now possible to obtain more exact information on the metallography and constitution of aluminum alloys. Experiments to this end have been in progress in the laboratories of the Aluminum Co. of America for some time and will be reported in the technical press as the various important alloy systems are completed.

This brief note on the microstructure of the aluminum-iron alloys has been chosen as the starting point in the study of the metallography of aluminum alloys of high purity because one or more constituents containing an appreciable amount of iron are present in the microstructure of all aluminum alloys of ordinary purity. It is therefore necessary to have a thorough knowledge of the structural appearance of aluminum containing iron with only the smallest traces of other elements.

[1] Metallurgist, Research Bureau, Aluminum Co. of America, New Kensington, Pa.

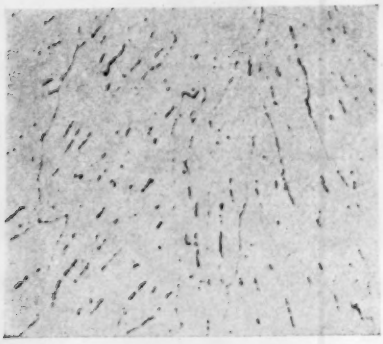
[2] F. C. Frary, "The Electrolytic Refining of Aluminum," *Transactions*, American Electrochemical Society, Vol. XLVII, p. 259 (1925).

The metal chosen for this series of experiments gave the following analysis: copper 0.016 per cent, iron 0.020 per cent, silicon 0.009 per cent, manganese nil, zinc nil; thus the impurities other than iron total less than 0.03 per cent. Armco iron wire of the following analysis: copper 0.042 per cent, silicon 0.005 per cent, manganese 0.022 per cent, carbon 0.013 per cent, phosphorus 0.005 per cent, sulfur 0.018 per cent, iron (by difference) 99.895 per cent, was added to this metal to form an iron-rich alloy containing 5.01 per cent of iron, copper 0.03 per cent, and silicon 0.01 per cent. This hardener was used for making all of the alloys discussed in this note with a single exception, which will be mentioned later.

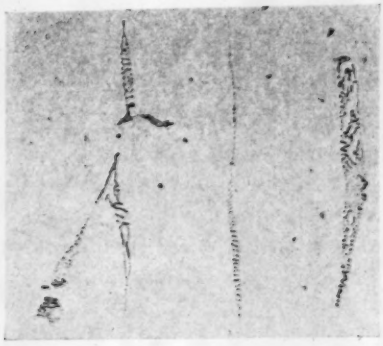
The iron-aluminum system has been investigated by a number of experimenters, but the high aluminum end did not receive careful attention until recently. It is now generally accepted that iron combines with aluminum to form the constituent $FeAl_3$ which forms a eutectic with the aluminum; the eutectic temperature is close to the melting point of pure aluminum and the eutectic contains only a few per cent of iron. It has been generally considered that iron is practically insoluble in aluminum.

The primary object of this paper is to illustrate the different modes of occurrence of the constituent $FeAl_3$ in aluminum of extremely high purity, and to this end twenty micrographs are presented. The areas illustrated are from cross-sections of bars cast in one of four different ways: (1) Bars $\frac{3}{4}$ in. in diameter by $2\frac{3}{4}$ in. long, cast vertically in a graphite mold; (2) bars $\frac{1}{2}$ by $\frac{5}{8}$ by 9 in., cast horizontally in an open iron mold with the $\frac{5}{8}$-in. dimension vertical; (3) bars $\frac{3}{4}$ in. in diameter by $2\frac{3}{4}$ in. long, cast vertically in green sand; and (4) blocks 1 in. in diameter by 1 in. long, slowly cooled in a graphite crucible in a gradient type furnace giving a 2° C. per minute rate of cooling through the solidification range. The graphite and iron molds were either cold or just slightly warmed. The melting was done in Acheson graphite crucibles to prevent contamination of the metal.

Figs. 1 (*a*) and (*b*) show the structure at magnifications of 100 and 500 diameters, respectively, of an alloy of the following analysis as cast from 940° C. in a cold iron mold: Iron 0.27 per cent, silicon 0.01 per cent, and copper 0.02 per cent. In a properly polished specimen the constituent appears bright with a slightly purple tinge. Fig. 9 shows the structure at 1000 diameters of an alloy containing 1.54 per cent of iron, cast in the graphite mold. Fig. 10 at 100 diameters magnification illustrates the appearance of the slowly cooled eutectic structure in an alloy containing 1.82 per cent of iron, which was cooled at the rate of 2° C. per minute from 700° C. Coarse primary needles

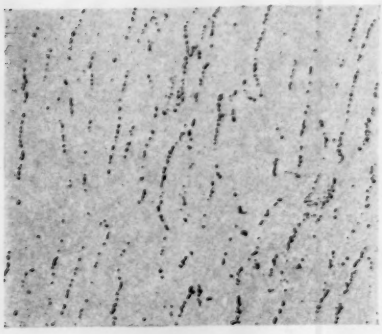


(*a*) Etched, 1 per cent hydrofluoric acid. (× 100)

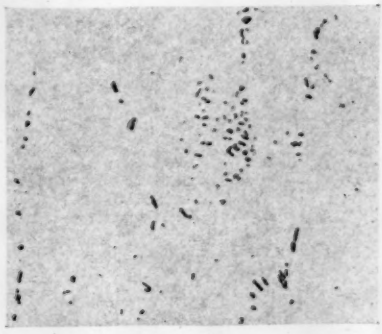


(*b*) Etched, 0.01 per cent hydrofluoric acid. (× 500)

FIG. 1.—Structure of Bar ½ by ⅝ by 9 in. Cast in Iron Mold. Analysis: Iron 0.27 per cent, Silicon 0.01 per cent, Copper 0.02 per cent.

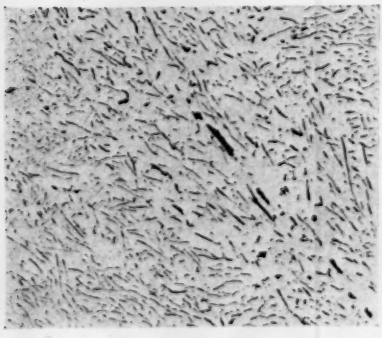


(*a*) (× 100)

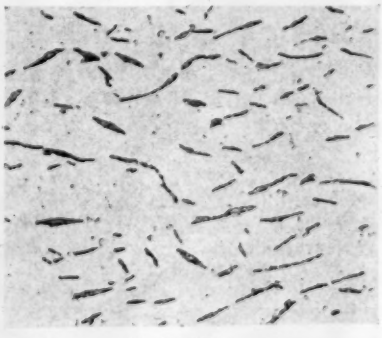


(*b*) (× 500)

FIG. 2.—Structure of Same Alloy as Fig. 1 but Annealed for 7 Days at 640 – 645° C. and Quenched. Etched, 1 per cent Hydrofluoric Acid.

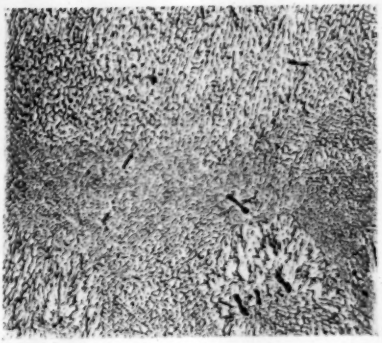


(*a*) (× 100)

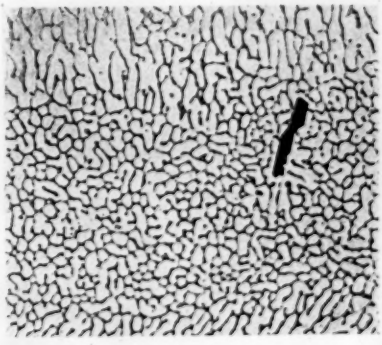


(*b*) (× 500)

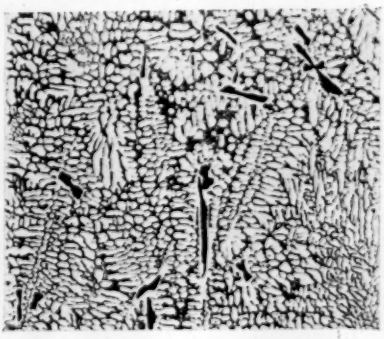
FIG. 3.—Structure of Bar ¾ in. in Diameter by 2¾ in. Cast in Green Sand, 2.2 per cent Iron (as mixed). Etched, 1 per cent Hydrofluoric Acid.



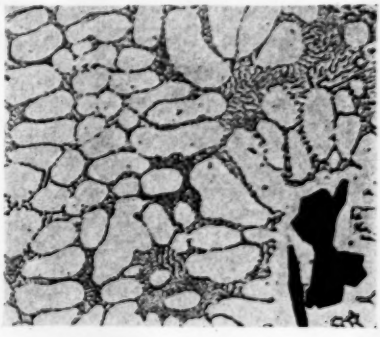
(*a*) Area near center of bar. (× 100)



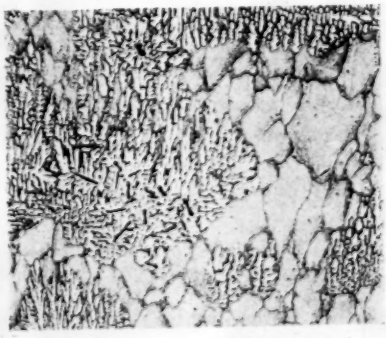
(*b*) Area near center of bar. (× 500)



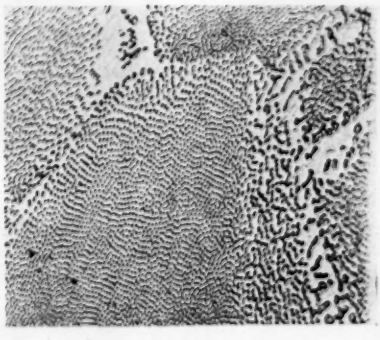
(*c*) Area mid-way between center and outside. (× 100)



(*d*) Area mid-way between center and outside. (× 500)



(*e*) Area near outside. (× 100)



(*f*) Area near outside. (× 1000)

FIG. 4.—Structure of Bar $\frac{3}{4}$ in. in Diameter by $2\frac{3}{4}$ in. Cast in Graphite Mold. Analysis: Iron 2.68 per cent, Silicon 0.01 per cent, Copper 0.02 per cent. Etched, 1 per cent Hydrofluoric Acid plus Nitric Acid.

of $FeAl_3$ had formed in the melt and settled to the bottom of the crucible so that an analysis of a sample taken from the same location as the micrograph showed only 1.69 per cent of iron. An alloy made up to contain 1.7 per cent of iron and similarly cooled did not show any primary needles at the bottom; it was not entirely homogeneous, however, for some coarse needles were found as well as some excess aluminum areas. An analysis of a sample taken from the same location as in the previously mentioned alloy showed 1.67 per cent of iron. It is evident, therefore, that the eutectic concentration may be taken as very close to 1.7 per cent of iron.

Preliminary experiments in this laboratory have indicated that the pure $FeAl_3$—aluminum eutectic temperature is close to 655° C. The National Physical Laboratory[1] has placed the eutectic concentration at 2 per cent of iron and has given its melting point as 648° C.

Fig. 3 shows the characteristic appearance of a sand-cast specimen of an alloy slightly in excess of the eutectic concentration, $\frac{3}{4}$ in. in diameter by $2\frac{3}{4}$ in. long. The alloy was made up to contain 2.2 per cent of iron but was not analyzed. Alloys slightly in excess of the eutectic concentration have a tendency to rather extreme segregation when chill cast, giving rise to very peculiar structural appearances in even small sections. For instance, Fig. 4 illustrates the variety of structures to be found in a $\frac{3}{4}$-in. diameter cross-section of an alloy containing 2.68 per cent of iron, silicon 0.01 per cent, and copper 0.02 per cent when cast in a cold graphite mold. This alloy is the exception previously mentioned in regard to the materials used in producing it. It was made from a 10-per-cent iron-rich alloy. Figs. 4 (*a*) and (*b*) show the structure at the center of the specimen at 100 and 500 diameters magnification, respectively. The band of extremely fine network which is shown running horizontally across Fig. 4 (*a*) is illustrated at higher magnification in Fig. 4 (*b*). This structure would indicate primary aluminum areas surrounded by a eutectic network, although primary particles of $FeAl_3$ are to be noted in both micrographs. Figs. 4 (*c*) and (*d*) illustrate the structure midway between the center and the outside at magnifications of 100 and 500 diameters, respectively. These micrographs show areas of excess aluminum surrounded by a eutectic network, which in places attains considerable width and the particles of $FeAl_3$ are readily discernible. Primary particles of $FeAl_3$ are also found scattered throughout the mass in various characteristic shapes. Figs. 4 (*e*) and (*f*) illustrate the structure found near the outside of the specimen at 100 and 1000 diameters, respectively. In Fig. 4 (*e*), areas of excess aluminum and primary

[1] Eleventh Report to the Alloys Research Committee, Inst. Mechanical Engrs. (1921).

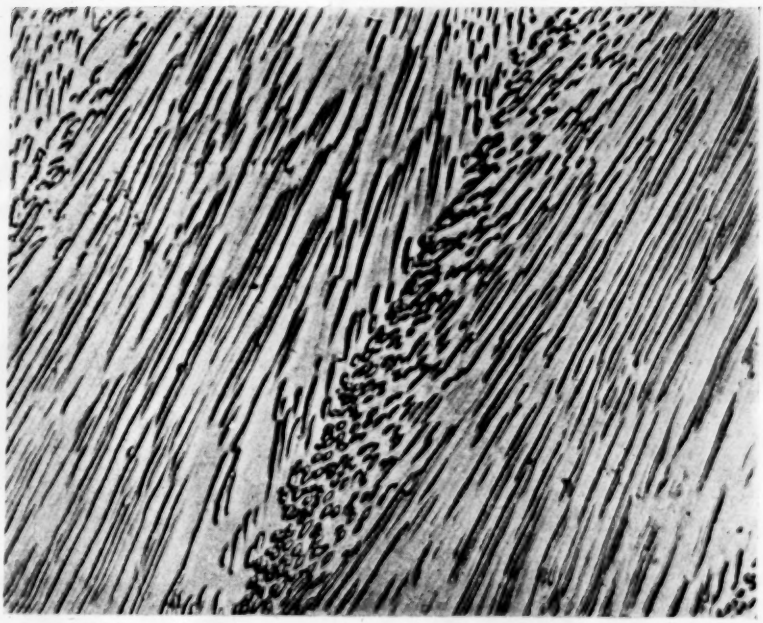


FIG. 5.—Same Specimen as Fig. 4, Showing Lengthwise Section of Needles. Etched, 1 per cent Hydrofluoric Acid. (× 2000)

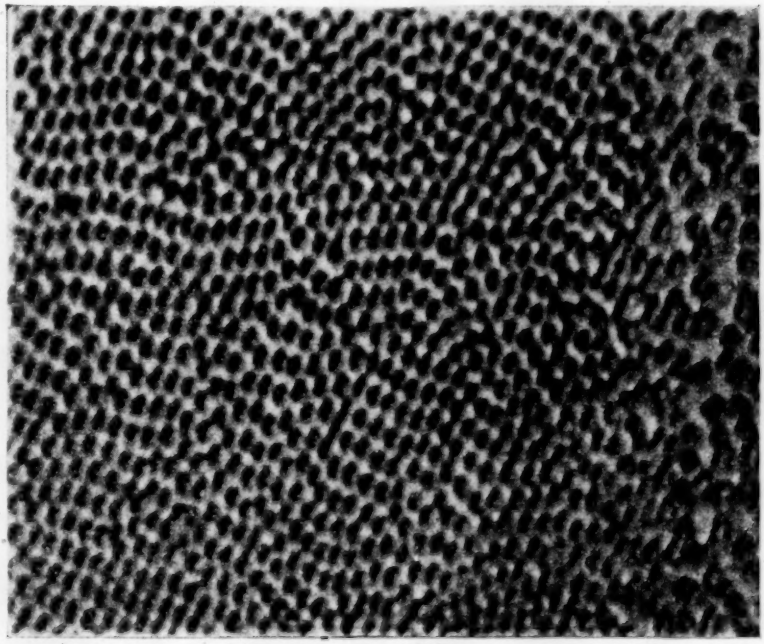


FIG. 6.—Same Area as Fig. 4 (*f*), Showing Cross-section of Needles. Etched, 1 per cent Hydrofluoric Acid. (× 5000)

particles of $FeAl_3$ are to be noted, just as in the previous micrographs, but in addition there are leaf-like areas which are totally unresolved at the 100 diameter magnification. One of these areas from the lower part of Fig. 4 (*e*) is shown at the higher magnification of Fig. 4 (*f*), and it will be noted that in the center of this area the individual $FeAl_3$ particles are resolved with difficulty, even at this magnification. The coarsening of the structure at the boundary of these areas is worthy of note. Figs. 5 and 6 are shown at higher magnifications in order to illustrate the exact nature of this fine structure. Fig. 5 includes several areas in which the particles of $FeAl_3$ are shown to be long, slender needles. It is unusual to obtain the lengthwise section of these needles because the polished surface must be parallel to the axis of a bundle of parallel needles in order to cut them lengthwise, whereas a number of planes with various angles of inclination to the axes of the needles will cut them so as to show the round or oval cross-section. Fig. 6 shows an area of Fig. 4 (*f*) at 5000 diameters magnification. The cross-sections of the individual needles are well resolved in this micrograph, and the fact that the cross-sections are oval instead of round indicates that the plane of the micrograph cut the needles at an angle rather than perpendicularly. The coarsening of the structure will again be noted at the right.

This extremely fine structure, which had never previously come to the author's attention, was discovered in January, 1924. It was of such striking interest that a number of experiments were undertaken to determine more of its nature. At F. C. Frary's suggestion a higher pouring temperature was tried and it was found that on remelting this same alloy and casting from 1100° C. in a cold graphite mold the structure of the specimen consisted almost entirely of these areas of fine structure except for a small spot in the center, which consisted of excess aluminum areas surrounded by a fine network similar to that shown in Fig. 4 (*b*). The structure at low magnification resembled the appearance of trees on a distant mountain side. No primary $FeAl_3$ was found in the cross-section examined. This procedure was repeated with alloys of higher iron content, and it was found possible to obtain the same fine structure with only a few groups of primary $FeAl_3$ particles in concentrations as high as 3.4 per cent of iron. However, if these same alloys were cast from a lower temperature even after being heated to 1100° C., or cooled more slowly after casting, as for instance, by casting in sand, large quantities of primary $FeAl_3$ were formed and the structure resembled that shown in Fig. 4 (*c*).

The structure of a 3-per-cent iron alloy cast from 940° C. in a cold graphite mold is illustrated in Fig. 7. This shows primary $FeAl_3$

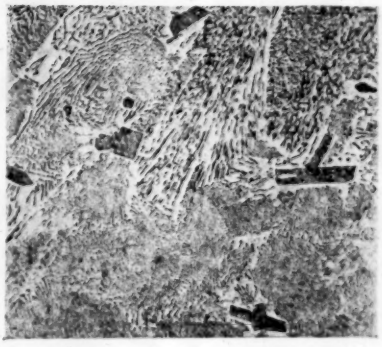


FIG. 7.—Structure of Bar ½ by ⅝ by 9 in. Cast in Iron Mold from 940° C., 3 per cent Iron (as mixed). Etched, 1 per cent Hydrofluoric Acid. (× 500)

FIG. 8.—Structure of Bar ½ by ⅝ by 9 in. Cast in Iron Mold from 1100° C., 3.48 per cent Iron (by analysis). Etched, 0.1 per cent Hydrofluoric Acid. (× 500)

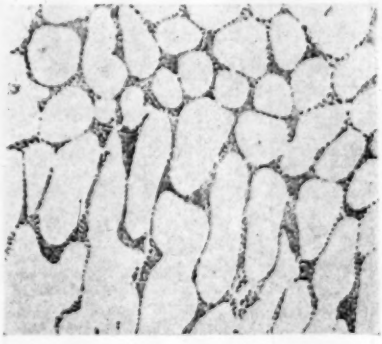


FIG. 9.—Structure of Bar ¾ in. in Diameter and 2¾ in. in Length Cast in Graphite Mold from 1100° C., 1.54 per cent Iron (by analysis). Etched, 1 per cent Hydrofluoric Acid. (× 1000)

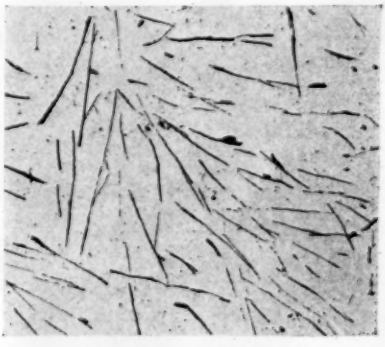


FIG. 10.—Bar 1 in. in Diameter by 1 in. Cooled at 2° C. per Minute from 700° C., 1.69 per cent Iron (by analysis). Etched, 1 per cent Hydrofluoric Acid. (× 100)

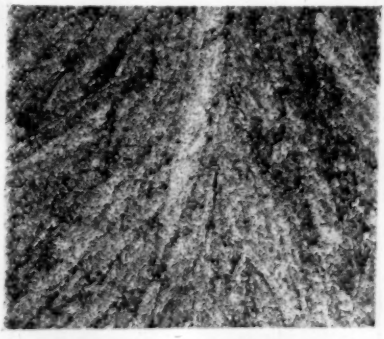


FIG. 11.—Structure of Bar ½ by ⅝ by 9 in. Cast in Iron Mold from 1100° C., 3.48 per cent Iron (by analysis). Etched, 1 per cent Hydrofluoric Acid. (× 100)

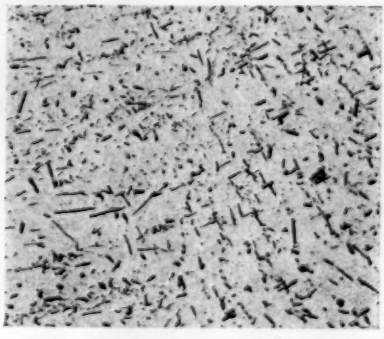


FIG. 12.—Section of Same Bar as Fig. 11, Annealed for 7 Days at 640–645° C., and Quenched. Etched, 1 per cent Hydrofluoric Acid. (× 100)

particles surrounded by pure aluminum areas with the balance of the structure a fairly fine eutectic. Fig. 8 shows an area in an alloy containing 3.48 per cent of iron cast in a cold iron mold from 1100° C. This structure consisted almost entirely of the fine eutectic areas, although after examining a number of sections, small groups of primary $FeAl_3$ were noted. This whole phenomenon is being subjected to a careful study, but the work has not progressed far enough at this time to draw any conclusions as to its exact nature. It is hoped to make this the subject of a later and more theoretical paper. It is evident that although the slowly cooled eutectic contains 1.7 per cent of iron yet it is possible under certain conditions to obtain a much finer structure with nearly twice the amount of iron. A high pouring temperature and rapid chilling are conditions which produce the latter structure. This structure as obtained by casting the 3.48-per-cent iron alloy in a cold graphite mold has a Brinell hardness of 49.6, whereas, the same alloy sand cast has a hardness of 33.3 (12.61-kg. load on a $\frac{1}{16}$-in. ball).

A number of specimens of these alloys cast in the iron mold were annealed in a copper block at a temperature of 640 to 645° C. for seven days and then some were quenched in cold water and others slowly cooled to room temperature. The annealing block and method employed will be discussed in more detail in a later paper. For the present, however, we are interested primarily in the effect of this annealing on the microstructure. It was found that with as little as 0.06 per cent of iron, 0.02 per cent of silicon and 0.02 per cent of copper, particles of the iron constituent were visible in both the quenched and slowly cooled specimens annealed at this temperature. Figs. 2 (*a*) and (*b*) may be compared with Figs. 1 (*a*) and (*b*) to illustrate the effect of annealing on the alloy containing 0.27 per cent of iron. It will be noted that the $FeAl_3$ has exhibited a pronounced tendency to coalesce. A more striking example is shown on comparing Figs. 11 and 12. Fig. 11 illustrates the structure at the center of a bar of rectangular cross-section, $\frac{1}{2}$ by $\frac{5}{8}$ in., cast in an iron mold from a temperature of 1100° C. The iron content of the alloy is 3.48 per cent. This structure consists almost entirely of the extremely fine eutectic which was not resolved at 100 diameters. Some few groups of primary $FeAl_3$ particles were found in different cross-sections of the bar. Fig. 12 illustrates a specimen cut from this same bar and quenched in cold water after the annealing, as previously described. The coalescence of the $FeAl_3$ particles in the form of straight needles is an example of diffusion in the solid state, and is striking in a system where the solubility is so low.

Etching Reagents.—The etching characteristics of the pure iron-aluminum alloys are of interest. All of the generally recognized etching reagents for aluminum alloys have been tried, but the author has obtained the greatest success by swabbing the specimen with a soft cotton swab and a solution of 1-per-cent hydrofluoric acid (made by adding 1 cc. of commercial hydrofluoric acid to 99 cc. of water). For the very fine structures to be photographed at high magnifications, a concentration of 0.1 or 0.01 per cent hydrofluoric acid has been used with good results. The hydrofluoric acid slightly attacks the aluminum solid solution and so removes any slight surface flow which may have been obtained even with the most careful preparation and the swabbing removes the solution products. If the etching is not carried beyond this point, the $FeAl_3$ retains its characteristic bright purplish color, but if the attack is continued for a longer period of time, the $FeAl_3$ is darkened at first and then colored either a brown or blue, depending on the conditions under which the etching is done. Generally the specimen is warmed by holding it under a stream of hot water before swabbing with the etching solution and it is then immediately washed off in the hot running water and blown dry. If swabbing is not used, a few drops of nitric acid may be added to prevent tarnishing the surface; it seems to have no other effect. A 10-per-cent solution of hydrofluoric acid darkens the $FeAl_3$ very rapidly and is not as satisfactory as the lower concentration. Sodium hydroxide has an effect very similar to that of the hydrofluoric acid, and satisfactory results will be obtained by swabbing with a 1-per-cent or 0.1-per-cent solution. The action of the 10-per-cent solution is far too rapid. The coloring of $FeAl_3$ by this reagent is very similar to that produced by hydrofluoric acid, but the results are not quite as satisfactory. The etch with 20-per-cent sulfuric acid at 70° C., which has been recommended as turning $FeAl_3$ black, was found to produce this effect in the pure aluminum-iron alloys; and it was likewise found that the ferric sulfate etch, recommended for distinguishing between $CuAl_2$ and $FeAl_3$, was satisfactory in that it did not color the $FeAl_3$. It was also found that the action of 25-per-cent nitric acid at 70° C. was practically nil after immersion for several minutes.

Acknowledgment.—In conclusion, the author desires to acknowledge with thanks the services of Mr. J. A. Nock and Mr. H. H. Richardson for assistance in the experimental work, and of Mr. George W. Wilcox for his excellent work in preparation of the specimens and the developing of negatives and prints, and to express appreciation to Mr. H. V. Churchill, under whose direction the chemical analyses were made, and to Mr. J. D. Edwards and Mr. C. S. Taylor for valuable suggestions.

DISCUSSION

Mr. McAdam.

MR. D. J. MCADAM, JR.[1]—The paper presented by Mr. Dix is of great interest. It shows the great advances that will be possible through the use of pure aluminum, in studying the microstructure and physical properties of aluminum alloys. The author has obtained excellent results in preparing and photographing his specimens.

The author's discussion of the eutectic concentration, however, has given the impression that he believes the eutectic concentration to vary with the casting conditions such as pouring temperature and rate of cooling. In the synopsis he says: "A marked difference in the apparent eutectic concentration as indicated by chill-cast and slowly cooled specimens is pointed out." On page 121 also he says: "Fig. 10 illustrates the appearance of the slowly cooled eutectic." On the other hand, he says on page 126 that Fig. 7 showing an alloy containing 3 per cent of iron, consists chiefly of "a fairly fine eutectic." He also says of Fig. 8, showing an alloy containing 3.48 per cent of iron: "This structure consisted almost entirely of the fine eutectic areas." A similar use is made of the word "eutectic" in connection with Fig. 11.

Now a comparison of Fig. 10 with Figs. 7, 8 and 11 will show that the author uses the word "eutectic" in a very broad sense to describe a type of microstructure. I believe that this broad use of the word eutectic is not advisable.

Not every microstructure that shows at some magnification a finely dispersed mixture of two constituents should be called an eutectic structure. In steel, for example, high magnification shows sorbite to consist of finely dispersed particles of cementite in ferrite. Absence of nuclei and with suitable rate of cooling may result in sorbite of much lower carbon content than the eutectoid percentage. With slower cooling, lamellar pearlite of eutectoid composition is obtained. With still slower cooling, "divorced" ferrite and cementite are obtained.

It seems possible that with aluminum-iron alloys under suitable cooling conditions, and with few nuclei for deposition of $FeAl_3$, structures analogous to sorbite may be obtained. With a slower rate of cooling, a true eutectic structure would be obtained, and with still slower cooling, divorced aluminum and $FeAl_3$.

[1] Metallurgist, U. S. Naval Engineering Experiment Station, Annapolis, Md.

Mr. McAdam.

The author by his annealing experiments has shown the ease with which "coalesence" of $FeAl_3$ or "divorce" of the constituents of a eutectic or hyper-eutectic complex occurs. On account of this great tendency to coalescence, it is probably difficult to regulate cooling rates of hypo- or hyper-eutectic alloys so as to obtain a true eutectic structure. More commonly a hypo- or hyper-eutectic complex is obtained or there is divorce of the constituents of the complex. Figs. 7, 8 and 11, therefore, probably represent an hyper-eutectic complex analogous to hypo-eutectoid sorbite. Figs. 2, 3, 10 and 12 show "divorced" $FeAl_3$. Figs. 4 and 9 represent intermediate stages. It seems probable that at least part of what the author calls "segregation" in the specimen shown in Fig. 4 is actually variation in the degree of coalescence of the two constituents.

One of the reasons for the difficulty of obtaining a true eutectic structure in aluminum-iron alloys is the great predominance of the aluminum in the eutectic. Whereas pearlite contains about 7 times as much ferrite as cementite, the aluminum-$FeAl_3$ eutectic contains 25 times as much aluminum as $FeAl_3$. This predominance of the aluminum favors the divorce of the constituents of the eutectic.

It seems probable, therefore, that there is only one eutectic and that the microstructure shown in some of the figures are not eutectics but hyper-eutectic complexes analogous to hypo-eutectoid sorbite.

I present this as a possible interpretation of the microstructures. As the author says, further study will be necessary to determine the correct interpretation.

Mr. Dix.

MR. E. H. DIX, JR.—I want to thank Mr. McAdam for his discussion. I should like to put emphasis on my use of the word "apparent" before eutectic concentration in the summary. I had hoped to dodge the theoretical point involved in this study and make the present paper merely an illustration of the constituents found in the aluminum iron alloys. For that reason I used "apparent" eutectic concentration in referring to alloys not in equilibrium. On page 124 the eutectic concentration is given as close to 1.7 per cent of iron and in this case I, of course, referred to the equilibrium eutectic. The other references given by Mr. McAdam concern eutectic structure either so stated or implied and in this case "eutectic" is used as an adjective to describe a structure which has the appearance commonly associated with the freezing of a eutectic mixture. It may be that this is a distinction without a difference but the intention was to describe a structure which has the appearance of a eutectic. This I believe is a common usage among metallographers. Naturally, we cannot reconcile our consciences with the phase rule and equilibrium diagrams and speak of different eutectic concentrations.

Mr. Dix. We rather feel that the formation of this very fine structure is largely influenced by the under-cooling of these alloys, producing a shower effect of the particles, and as I noted we hope to give this more careful consideration and perhaps at some later date submit a more theoretical paper.

Mr. McAdam. MR. MCADAM *(by letter)*.[1]—In my oral discussion of this paper I suggested that the shifting of the position of the apparent eutectic by rapid cooling of aluminum-iron alloys is due to "surfusion." There is clear internal evidence in Mr. Dix's paper that he has had in mind the idea of surfusion although he has not explicitly stated it. In later conversation also, Mr. Dix showed that he was familiar with the paper of Portevin[2] in which the effect of surfusion on "eutectics" is discussed.

As stated in my oral discussion the author uses the word eutectic not only in its original sense to describe an alloy of lowest melting point, an alloy at whose melting point solid and liquid phases can exist in equilibrium at constant temperature, a non-variant system. He also uses the word eutectic to describe the microstructure of binary alloys which are not eutectic alloys. Although in this ambiguous use of the word "eutectic" the author has distinguished company,[2] the writer believes that such use of the word will often cause confusion.

It seems probable that the apparent eutectic structures, obtained by the author when aluminum-iron alloys containing as much as 3.5 per cent of iron are cooled rapidly, are due to surfusion. It can be shown that, if the tendencies to surfusion with reference to the two microconstituents differ, the composition of the apparent eutectic obtained by rapid cooling may differ from that of the true eutectic. This has been pointed out by Portevin.[2] This effect of surfusion is illustrated by the accompanying Fig. 1 which represents qualitatively the possible variation in the position of the apparent eutectic with variation in the rate of cooling.

In Fig. 1 the lines AB and BC represent the liquidus of the equilibrium diagram for the aluminum-iron alloys, although the direction of the line BC may not be exactly correct. The lines AB and BC represent equilibrium between liquid and solid aluminum and between liquid and $FeAl_3$ respectively. The lines $A'B'$ and $B'C'$ represent qualitatively the boundaries between liquid and solid when the alloy is cooled rapidly. The surfusion tendency of aluminum is probably slight, whereas the surfusion tendency of the compound

[1] Discussion printed by permission of the Secretary of the Navy.

[2] Albert M. Portevin, "The Structure of Eutectics," *Journal*, Inst. of Metals, 1923, No. 1, Vol. XXIX.

Mr. McAdam.

$FeAl_3$ is evidently great. The line $A'B'$ has therefore been drawn only slightly below the line AB and the line $B'C'$ has been drawn considerably below the line BC. The junction B' of the line $A'B'$ and $B'C'$ would represent the temperature at which the apparent eutectic is formed and its composition for a definite rapid rate of

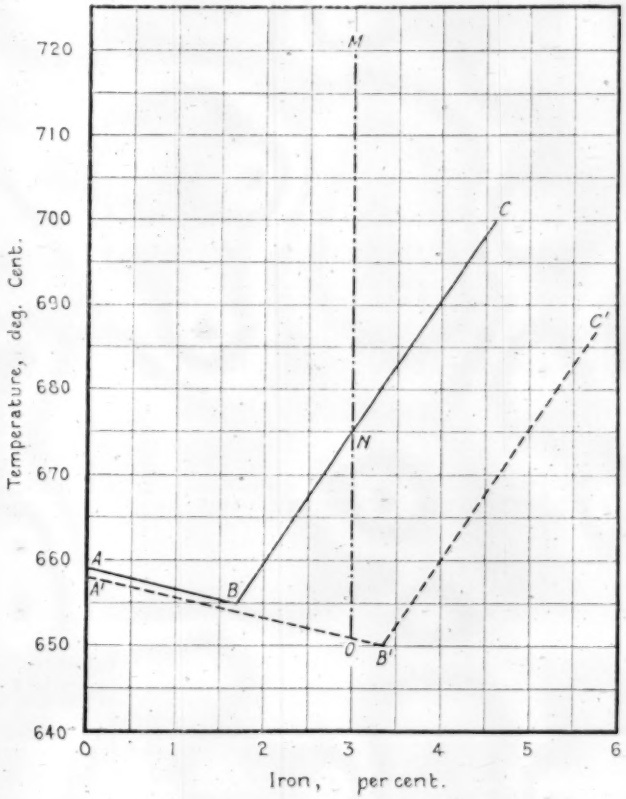


FIG. 1.—Effect of Surfusion on the Composition of the Apparent Eutectic Structure of Aluminum-Iron Alloys.

cooling. As represented in the figure, the apparent eutectic has about twice the iron content of the true eutectic alloy.

The presence of aluminum dendrites in rapidly cooling hyper-eutectic alloys, such as the alloy shown in Fig. 4 of the author's paper, is readily explained by means of Fig. 1. If a hyper-eutectic alloy, represented in temperature and composition by the point M in Fig. 1 be rapidly cooled, $FeAl_3$ does not form at a temperature represented

Mr. McAdam.

by the point N. The liquid remains surfused until a temperature represented by the point O is reached, at which temperature aluminum crystals appear. The presence of a few crystals of pro-eutectic $FeAl_3$ is due to the presence of a few scattered nuclei of this phase in the liquid before in cooling the point O is reached. Solid aluminum separates as the temperature and composition change from O to B'. At B' both aluminum and $FeAl_3$ would separate together to form a hyper-eutectic complex.

The composition of the apparent eutectic caused by surfusion varies with the rate of cooling. By varying the rate of cooling of aluminum-iron alloys it should be possible to obtain apparent eutectic structures the composition of which varies from 1.7 to more than 3.5 per cent of iron as illustrated by Fig. 1.

The conspicuous variation in the composition of the apparent eutectic with variation in the rate of cooling is due in part to the nearly horizontal direction of the line AB as shown in Fig. 1. Because of the nearly horizontal direction, even slight surfusion of the $FeAl_3$ would cause relatively great change in the apparent eutectic composition. Similar instability in composition of the apparent eutectic will probably be found not infrequently in other alloys whose equilibrium diagram contains a similar slightly sloping liquidus.

Portevin[1] as well as the author designates such hypo- and hyper-eutectic complexes as "eutectics." It seems advisable, however, that the term "eutectic" be restricted to its original meaning and that another term be used to designate hyper- and hypo-eutectic structures that consist of intimate mixtures of two microconstituents. If the same term is to be used, however, great care should be taken each time the word is used to avoid misunderstanding of its meaning.

The effect of surfusion on the microstructure of aluminum-iron alloys is similar to its effect on aluminum-silicon alloys. As pointed out by Jeffries and Archer,[2] with rapid cooling of aluminum-silicon alloys the microstructure is profoundly modified and the position of the apparent eutectic is changed. By treatment with sodium or some of its compounds, the alloy is "modified" so that surfusion occurs even when the rate of cooling is comparatively slow.[3, 4, 5] The alloy thus modified has the miscrostructure of the normal rapidly cooled

[1] Albert M. Portevin, "The Structure of Eutectics," *Journal*, Inst. of Metals, 1923, No. 1, Vol. XXIX.

[2] Zay Jeffries and R. S. Archer, "The Science of Metals," pp. 325–326.

[3] Zay Jeffries, "Aluminum-Silicon Alloys," *Chemical and Metallurgical Engineering*, Vol. 26, No. 16, April 19, 1922.

[4] J. D. Edwards, "Properties and Manufacture of Aluminum-Silicon Alloys," *Chemical and Metallurgical Engineering*, Vol. 27, No. 13, Sept. 27, 1922.

[5] J. D. Edwards and R. S. Archer, "The New Aluminum-Silicon Alloys," *Chemical and Metallurgical Engineering*, Vol. 31, No. 13, Sept. 27, 1924.

alloy. Edwards and Archer[1] attribute this effect of sodium to mechanical obstruction by sodium films to the growth of silicon crystals. Whether or not this explanation is correct, the phenomena are similar to those of surfusion. Mr. McAdam.

Additional papers by Mr. Dix on aluminum alloys will be awaited with interest.

MR. E. H. DIX, JR. (*author's closure by letter*).—The theoretical aspects of this study were omitted from the original paper for two reasons: (1) Since the paper was presented to the Society as an aid in identifying the iron constituents which occur in commercial aluminum alloys, it seemed undesirable, owing to the limited space allotted to the paper, to include anything which did not tend directly towards this end and (2) Certain of the data obtained did not seem to be satisfactorily explained by the usual effects of undercooling.[2] On page 128 it is stated that "although the slowly cooled (equilibrium) eutectic contains 1.7 per cent of iron, yet it is possible under certain conditions to obtain a much finer structure with nearly twice the amount of iron. A high pouring temperature and rapid chilling are conditions which produce the latter structure." The effect of the rapid chilling is readily explained by the well-known effects of undercooling, but the effect of the high pouring temperature is not so easily explained. For instance, it was shown that 940° C., which is well above the point of primary separation of $FeAl_3$, was yet not sufficiently high to give the maximum effect such as was obtained by pouring from 1100° C. Mr. Dix.

Mr. McAdam has called attention to the effect of the slight inclination from the horizontal of the line representing the separation of primary aluminum, showing that a few degrees of undercooling of the eutectic point produces a large change in the apparent eutectic concentration. Our attention[3] had previously been called to the very abrupt slope of the liquidus line representing the primary separation of $FeAl_3$. The National Physical Laboratory[4] as a result of experimental data has given a somewhat greater slope to this line than that assumed by Mr. McAdam. Since this slope is so very abrupt, the amount of $FeAl_3$ separating at any given temperature is very small and this introduces experimental difficulties, which leave considerable doubt as to the exact position of the line. The tendency would be for thermal analysis to indicate the primary separation as occurring at a temperature somewhat lower than the true temperature. In this

[1] J. D. Edwards and R. S. Archer, "The New Aluminum-Silicon Alloys," *Chemical and Metallurgical Engineering*, Vol. 31, No. 13, Sept. 27, 1924.

[2] See Desch's "Metallography," edition of 1910, as well as the later textbooks of Gulliver, Tammann and others.

[3] J. D. Edwards, private communication.

[4] Eleventh Report to the Alloys Research Committee, The Institution of Mechanical Engineers.

Mr. Dix. way there is some uncertainty as to the exact temperature interval between a pouring temperature of, say, 940° C. and the point of primary separation for a given alloy, as, for instance, the 3.48-per-cent-iron alloy. However, from the best information available, this interval seems to be in the neighborhood of 200 to 250° C., which should certainly be ample, if the undercooling during freezing is the only condition to be considered. However, it would seem that the rate of cooling of the liquid above the liquidus line might have some effect on the formation of nuclei when freezing does start, This effect has never been subjected to satisfactory experimentation to the author's knowledge, and it was hoped to carry out some further experiments on this and other aluminum systems before publicly discussing the theory of the phenomena.

ACCELERATED CORROSION TESTS ON BARE OVERHEAD ELECTRICAL CONDUCTORS

By Frank F. Fowle[1]

Synopsis

This paper describes a laboratory method of simulating the corrosive influences of unfavorable outdoor atmospheres found in land regions of the city or industrial types, under moderate but not excessive acceleration, as affecting the usual types of bare or uninsulated overhead electrical conductors, including galvanized iron and steel. The test apparatus comprised a closed box about 16 ft. long, 5 ft. wide and 4 ft. high, with associated equipment for introducing warm dry air, humid air, dilute bituminous coal smoke, and fine water spray, at will. These four conditions were introduced in the order named, with a total cycle of 8 hours, repeated approximately 800 times. The specimens were 14 ft. long, arranged in a horizontal plane in the center of the box, with baffles causing the gases to pass across them several times. There were 21 specimens of wire and strand, including copper, aluminum, copper-clad steel, galvanized iron and galvanized steel; also a new type of stranded conductor, consisting of six outer wires of galvanized steel and a center wire of tinned or galvanized copper. These specimens were tested before exposure to determine the tensile strength, elongation, chemical analysis, weight of zinc coating, and number of dips under the Preece test. Periodical examinations were made of the specimens about every 100 cycles and their condition was carefully noted in the test log. The test was continued until all zinc coatings on exposed wires had failed, stopping after 793 cycles. Certain specimens were measured for tensile strength, loss of weight and decrease of diameter, at the end of the test. A summary of results is given in Table V. The method is regarded as reliable for obtaining comparative results, but at present no basis has been developed for equating the exposure into probable life in overhead service. An outdoor rack, with similar specimens, is now under test, but has not advanced sufficiently to report.

Objective

The original motive in conducting the tests described in this paper was to determine what effect, from the viewpoint of resistance to atmospheric corrosion, would result from removing the center wire of a 7-wire galvanized steel strand and substituting a galvanized or tinned copper wire. Considerable prior work had been done by the author on such a type of copper-steel strand to determine its electrical and mechanical properties as an electric power conductor and the results were thought sufficiently encouraging to justify an investiga-

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tion of the resistance to corrosion. In planning this investigation it was finally decided that the results would be more useful if all the usual types of bare (uninsulated) overhead electrical conductors were included and therefore specimens of several commercial grades of galvanized iron and steel wire and strand, bare copper wire and strand, aluminum wire, steel-reinforced aluminum strand, and copper-clad steel wire and strand, were added to the list.

The obvious purpose of replacing the center wire of a steel strand with copper wire is to improve the electrical conducting properties of the strand as a whole. The method of securing this result is outside the scope of this discussion but the presence of uncoated copper adjacent to steel wires would be expected to cause a marked increase in the normal rate of corrosion of the latter. Therefore this center wire is coated with tin or zinc and the principal specific object of this investigation was to determine whether or not steel strands with a copper center wire coated with tin or zinc would deteriorate any faster due to corrosion than the normal all-steel strand.

SELECTION OF METHOD

After reviewing the earlier methods for testing the durable properties of a zinc coating, in a comparatively brief operation, it was felt that none of them simulated the corrosive influences usually present in the atmosphere, as found in the great majority of localities about the country. It is well known that aside from the ordinary effects of atmospheric oxygen, changes in moisture conditions, and the drying actions of sun and wind, the chief corrosive agent in unfavorable atmospheres is the combustion product of bituminous coal, in both gaseous and finely divided solid form, containing dilute compounds of sulfur and of carbon, and often accompanied by atmospheric dust consisting of both cinder and ash. These combined influences rapidly attack bare iron and the softer varieties of steel, although the presence of certain small ingredients in the metal will lessen the rate of attack, but not prevent it. According to general experience with all of the ordinary types of protective zinc coatings heretofore common in the arts, the coating itself is slowly attacked by these same corrosive agents and in time will be reduced sufficiently to expose the iron base beneath.

None of the previous methods known to the author appeared to approach even remotely the actual service conditions mentioned above and furthermore it was suspected that perhaps some of the methods in previous use, such as the Preece test and the salt spray test, did not afford a reliable index of the comparative service merits

of any particular zinc coating. Therefore it was determined to experiment with a method which would incorporate the more active corrosive agents commonly prevalent in unfavorable atmospheres, as before mentioned, and submit specimens of various common types of overhead uninsulated conductors to these influences, systematically applied in repeated cycles of short duration. It was expected that this would furnish an accelerated life test, without the use of unnatural or unduly concentrated corrosives, but sufficiently expeditious to reach conclusions within a comparatively small fraction of the average life in actual service.

DETAILS OF METHOD

The method finally developed for this test consisted in placing specimens of wire and strand, about 14 ft. in length, in a closed box where they were exposed to successive cycles of corroding conditions in the following order:

Warm dry air	3.0 hours
Warm moist air	1.5 "
Air charged with bituminous coal smoke	3.0 "
Heavy fine water spray	0.5 "
Total	8.0 "

This cycle was repeated continuously day and night, except Sundays and holidays, with only an occasional interruption. The samples were taken out and examined at intervals, as later explained. The function of the 3-hour warm-air period was to secure thorough drying of the samples and the box after the water spray at the end of the previous cycle. The moist-air period of 1.5 hours was intended to simulate a normal atmospheric condition which prevails much of the time. Bituminous coal smoke for an interval of 3 hours, followed by one-half hour of spray or rain, introduced the chief corrosive agents.

DESCRIPTION OF APPARATUS

A general view of the apparatus is shown in Fig. 1 and an isometric drawing in Fig. 2. Further details and overall dimensions are given in Fig. 3. The box was 16 ft. long, 5 ft. wide and 4 ft. high, constructed of wood and lined with tarred roofing paper. It was mounted on a platform merely as an incidental matter, to conserve useful space in the laboratory. Access to the interior was provided by making the cover of the box removable, by attaching it to counterweighted overhead ropes. The entire apparatus was located in the Electrical Testing Laboratories in New York City.

Circulation.—As indicated in Figs. 1 and 2, the box was provided with inlet and discharge pipes, connected at opposite ends. The inlet was at the left-hand end of the box in Fig. 2, and the outlet at the other end. Gentle circulation of air and gas was maintained by a small exhaust fan at the outlet, connected as an ejector in order to avoid corroding the fan itself. The transverse baffles indicated in

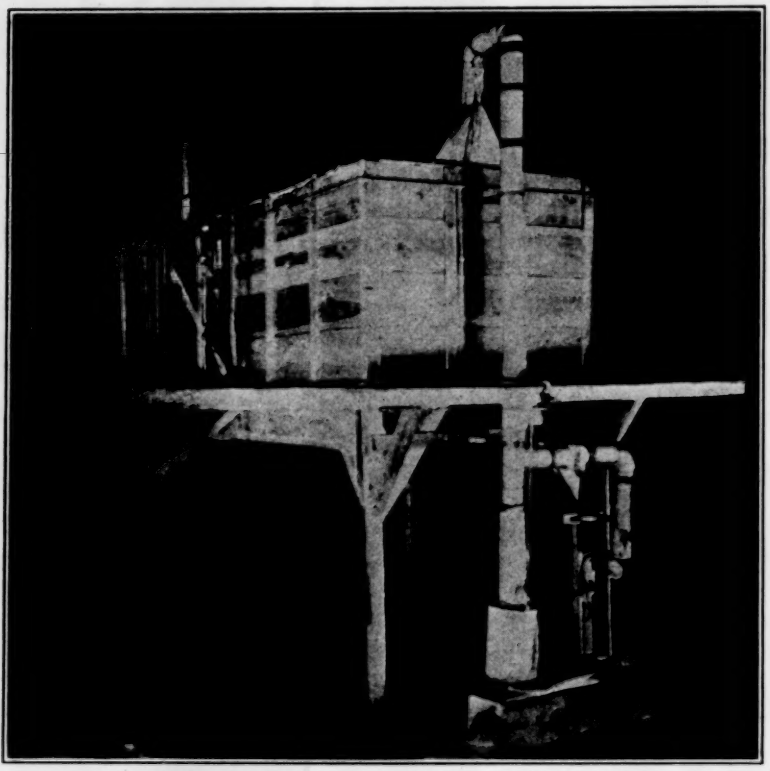


FIG. 1.—General View of Corrosion Box.

Figs. 2 and 3 caused the circulation to pass repeatedly across the test specimens.

Humidity.—Dry and humid air were conditioned in the apparatus shown in the foreground of Fig. 1 and at the left-hand end of the box in Fig. 2. This consisted of an open-bottom sheet-metal cylinder, attached at the top to the intake pipe leading to the corrosion box, with the bottom placed immediately above a gas-heated iron plate. Warm dry air was produced by drawing normal air from the room into this cylinder, across the hot plate. Moisture-laden air was

produced by allowing water to drip in sufficient quantity on the hot plate. Wet and dry bulb thermometers were placed inside the corrosion box to observe the temperature and humidity.

Smoke Generator.—Smoke-laden air was obtained by burning bituminous coal in the receptacle or "stove" shown at the right of the air heater in Figs. 1 and 2. It consisted essentially of a cylinder of pipe, opening into a branch of the intake pipe to the corrosion box,

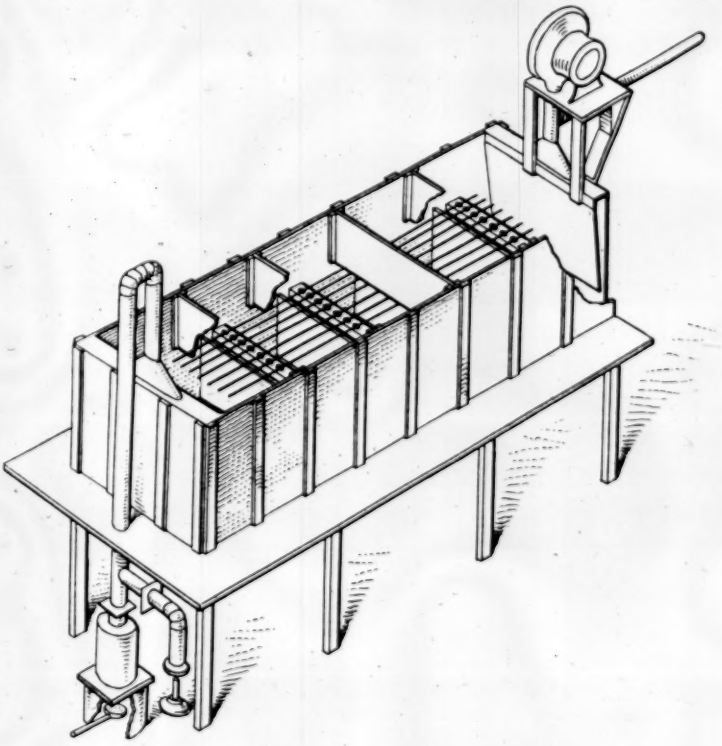


FIG. 2.—Isometric View of Corrosion Box.

and provided with a grate and a Bunsen burner. The rate of combustion was controlled by regulating the gas flame. When it was desired to pass smoke-laden air into the box, the air heater was cut off by closing the damper; and similarly, when dry or moist air was being drawn into the box, the "stove" was cut off by a damper. The necessary draft was induced by the fan before mentioned. The constituents of the smoke-laden air were determined by a gas analysis.

Water Spray.—The condition of outdoor rain was simulated by a fine water spray from nozzles arranged around the upper sides of

the box, so that the specimens received a thorough drenching from above. Drain connections in the bottom of the box carried away the waste water. The nozzles were adjustable and all were connected to a small supply pipe controlled by a single valve, connecting with the city main.

Position of Test Specimens.—The test specimens, initially about 14 ft. in length, were supported on small brown porcelain insulators with top grooves. These insulators were mounted on wooden pins, in turn supported by wooden cross-pieces spanning from side to side of the box, as can be seen from the skeleton sectional view in Fig. 3. These exposed wooden parts were protected by a bituminous paint.

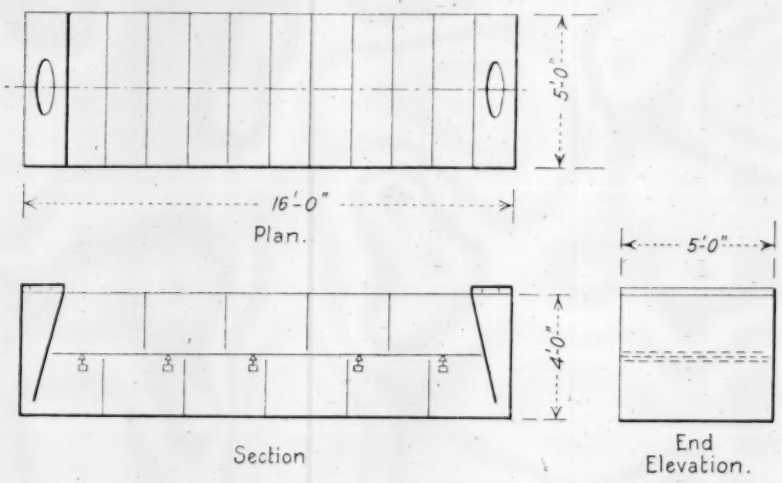


FIG. 3.—Plan, Section and End Elevation of Corrosion Box.

The specimens were made reasonably straight and permitted to rest in the grooves on the tops of the insulators, without any fastenings or tie-wires. As a whole the specimens were in a horizontal plane about midway between the top and the bottom of the box, as seen in Figs. 2 and 3. The upper baffles were of course removable, to permit of access to the specimens. Precautions were taken to avoid corrosion due to electrolytic action resulting from possible contact of the specimens with each other or with any other objects. Each specimen was identified by a small brass tag, loosely attached to the specimen by a small wire of the same material. There was one glass window in the side of the box and an incandescent lamp within, to provide a general view of the specimens while the test was in operation. Thus every effort was made to keep the interior of the box and the supports in a

chemically inert condition and hence avoid any effects except those specifically sought for. As a whole the construction of the box was rather obviously of temporary character, but it appeared to serve its initial purpose. In constructing a similar apparatus for permanent or extended use, it would be desirable to employ a sheet lead lining, with a water-seal connection between the cover and the side walls.

TABLE I.—LIST OF SAMPLES OF ELECTRICAL CONDUCTORS TESTED.

Identification Mark	Description or Commercial Name	Diameter or Gage of Wire or Complete Strand
1-A	Bare aluminum wire	0.167 in.
1-B	Bare steel-reinforced aluminum strand	4/0 A. W. G
2-A	Bare hard-drawn copper wire	0.165 in.
2-B	" " " " " strand	3/0 A. W. G.
3-A	Bare hard-drawn copper-clad steel wire	6 A. W. G.
3-B	" " " " " " strand	4/0 A. W. G
4-A	Double-galvanized Extra High Strength steel strand, with copper center[a]	$\frac{3}{8}$ in.
4-B	" " Siemens-Martin steel strand, with copper center[a]	"
4-C	" " High-Strength steel strand, with copper center[a]	"
4-D	" " High-Strength steel strand, with copper center[b]	"
5-A	Bare ingot iron wire	9 B. W. G.
5-B	Double-galvanized ingot iron wire	"
6-A	" " Best Best iron wire	8 B. W. G.
6-B	" " Steel wire	"
6-C	" " Extra Best Best iron wire	"
6-D	" " Swedish iron wire	"
6-E	" " Siemens-Martin steel strand	$\frac{7}{16}$ in.
6-F	" " Steel strand	"
6-G	" " High-Strength steel strand	"
6-H	" " Extra High-Strength steel strand	"
6-I	Bare Swedish iron wire	8 B. W. G.

[a] Tinned hard-drawn copper wire in the center of the strand.
[b] Galvanized copper wire in the center of the strand.

DESCRIPTION OF SPECIMENS TESTED

Although the primary object of the investigation was the deter mination of the behavior of galvanized wires and strands, and mor particularly the copper-center steel strand, samples of all commercial types of uninsulated conductor materials used for overhead electric transmission were included, for the sake of obtaining more comprehensive comparative results. In all there were 21 samples, which are listed in detail in Table I. Each different numeral in the identification mark denotes a different manufacturer and each different letter denotes a separate specimen. All specimens except the series 4-A

TABLE II.—TENSION TESTS ON SPECIMENS AS RECEIVED.

Identification Mark	Material	Diameter, mils	Breaking Load, lb.	Tensile Strength, lb. per sq. in.	Elongation in 10 in., per cent
1-A	Aluminum wire	167.6	577	26 100	3.8
1-B	Steel center wire	188.5	4 880	175 000	6.1
1-B	Aluminum outer wire	186.8	561	20 500	2.9
2-A	Copper wire	165.6	1 337	62 100	1.0[a]
2-B	Copper individual wires	155.7	1 173	61 650	1.6[a]
3-A	Copper-clad steel wire	164.3	2 030	94 500	1.5
3-B	Copper-clad steel individual wires	174.7	2 073	86 000	2.2
4-A	Copper center wire	128.8	790	60 700	2.4
4-A	Steel individual wires	120.8	2 230	194 750	4.1
4-B	Copper center wire	129.7	813	62 700	1.9
4-B	Steel individual wires	119.9	869	77 050	18.6
4-C	Copper center wire	129.0	835	63 800	2.8
4-C	Steel individual wires	120.2	1 824	160 750	5.3
4-D	Copper center wire	128.5	472	36 400	30.9
4-D	Steel individual wires	112.1	1 834	163 750	5.3
5-A	Ingot iron wire	146.6	732	43 300	24.9
5-B	Ingot iron wire	149.2	935	53 500	15.0
6-A	Best Best iron wire	162.8	1 072	51 500	17.3
6-B	Steel wire	165.5	1 304	60 700	15.2
6-C	Extra Best Best iron wire	162.5	1 061	51 200	16.3
6-D	Swedish iron wire	164.3	1 075	50 100	13.1
6-E	Siemens-Martin steel individual wires	144.1	2 143	131 400	9.1
6-F	Steel individual wires	146.5	994	59 000	13.3
6-G	High-Strength steel individual wires	144.5	2 819	172 150	5.7
6-H	Extra-High-Strength steel individual wires	146.4	3 415	203 100	6.6
6-I	Swedish iron wire	164.2	880	41 000	23.4

[a] Measured on a length of 60 in.

TABLE III.—CHEMICAL ANALYSIS OF IRON AND STEEL IN TEST SPECIMENS AS RECEIVED.

All values are given in per cent

Identification Mark	Carbon	Manganese	Silicon	Copper	Sulfur	Phosphorus
1-B	0.564	0.580	0.133	0.038	0.034	0.064
3-A[1]	0.116	0.017	0.061	0.208	0.034	0.014
3-B[1]	0.090	0.070	0.047	0.153	0.029	0.010
4-A	0.612	0.994	0.167	0.028	0.057	0.038
4-B	0.384	0.652	0.026	0.019	0.048	0.052
4-C	0.580	1.080	0.143	0.012	0.040	0.036
4-D	0.560	1.075	0.120	0.014	0.035	0.037
5-A	0.006	0.014	0.016	0.041	0.034	0.011
5-B	0.002	0.028	0.016	0.051	0.044	0.011
6-A	0.060	0.224	0.010	0.060	0.032	0.026
6-B	0.160	0.565	0.030	0.124	0.030	0.036
6-C	0.054	0.044	0.024	0.072	0.026	0.019
6-D	0.066	0.015	0.036	0.017	0.014	0.057
6-E	0.590	0.545	0.134	0.086	0.048	0.062
6-F	0.040	0.632	0.026	0.078	0.051	0.037
6-G	0.570	0.520	0.119	0.107	0.056	0.033
6-H	0.450	0.860	0.200	0.230	0.047	0.059
6-I	0.005	0.012	0.016	0.059	0.017	0.051

[1] The outer copper coating on these specimens was carefully removed by filing before making the analysis.

to 4-D, inclusive, were purchased in the open market without disclosure of the purpose, but the four copper-center steel strands were specially manufactured for test purposes, although the individual galvanized steel wires in these strands were ordinary product.

PRELIMINARY EXAMINATION OF TEST SPECIMENS

The test specimens enumerated in Table I were delivered to the Electrical Testing Laboratories early in the summer of 1923 and were then subjected to certain preliminary tests as given below.

Tension Tests.—Determinations of diameter, breaking load, tensile strength and total elongation were carried out by the usual

TABLE IV.—TESTS OF ZINC COATINGS;[1] SPECIMENS AS RECEIVED.

Identification Mark	Weight of Coating, oz. per sq. ft.	Number of Dips in Preece Test[2]		
		Without Failure	Failure at Point of Abrasion	Failure Elsewhere
1-B	....	7	8	..
4-A	1.09	4	5	6
4-B	1.22	4	5	6
4-C	0.86	3	4	5
4-D	0.94	3	4	5
5-B	1.05	4	..	5
6-A	1.02	7	..	8
6-B	1.15	5	..	6
6-C	0.82	6	..	7
6-D	1.28	6	..	7
6-E	0.97	5	..	6
6-F	1.15	3	4	5
6-G	1.02	3	4	5
6-H	1.18	3	4	5

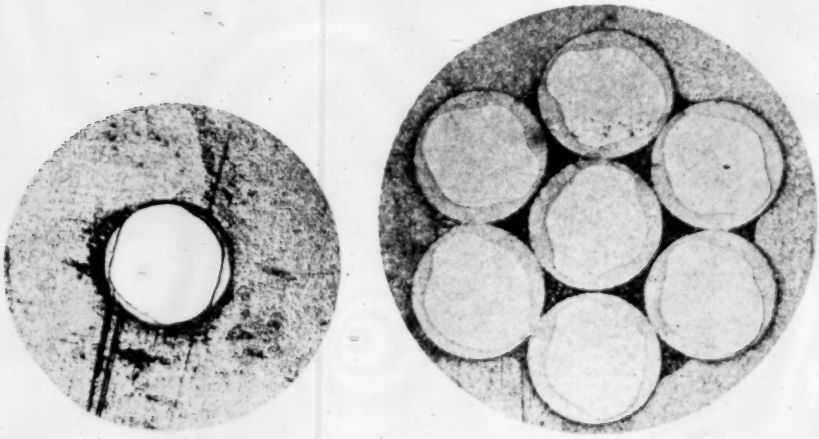
[1] The tin coatings on the copper wires in specimens 4-A and 4-B each failed on the third dip, at a point of abrasion; 4-C and 4-D failed likewise, on the second dip.

[2] The Preece Test is described in the Appendix to the Society's Standard Methods of Determining Weight of Coating on Zinc-Coated Articles (A 90 - 24), 1924 Book of A.S.T.M. Standards.

procedure and the results are summarized in Table II. These tests serve to identify certain properties of the test specimens, and by comparisons with corresponding determinations made at the end of the corrosion test, they serve further to show the effects of corrosion on the tensile properties. All tension tests were made on individual wires.

Chemical Analysis.—It was considered very desirable to make chemical determinations of the usual constituents of the iron and steel in the test specimens. This was carried out by standard methods and the results are summarized in Table III. It is interesting to note that every specimen in Table III had at least a small content of copper and in three instances, 3-A, 3-B and 6-H, this was appreciable.

Tests of Coatings.—Out of 21 samples used in this investigation, 14 contained one or more wires of iron or steel which had protective coatings, in most instances of zinc. These coatings were tested for initial condition in two ways, one by stripping and ascertaining the weight per unit area, and the other by the Preece test. All galvanized wires were tested in accordance with the standard specifications of the American Telephone and Telegraph Co.; the tinned wires were tested in accordance with the requirements of the Society's Standard Specifications for Tinned Soft or Annealed Copper Wire for Rubber Insulation (B 33 - 21).[1] The results are summarized in Table IV. It is notable that some of the specimens did not come up to the require-



Wire Strand

FIG. 4.—Copper-Clad Steel Cross-Sections.

The wires were embedded in a matrix of low-melting-point alloy for convenience in developing the cross-sections.

ment of four one-minute dips, but showed up pretty well on the corrosion test, whereas some specimens which withstood four or more dips did not stand up as long in the corrosion box. Another interesting point is the absence of any linear relation between the weight of the coating and the number of dips before failure in the Preece test. For example, 6-H with a weight of 1.18 oz. per sq. ft. withstood only three dips without failure, whereas 6-A with 1.02 oz. per sq. ft. withstood seven dips. Various other comparisons might be cited, some tending to confirm this observation and others tending to disprove it; but on the whole the absence of any definite relation is rather obvious, although of course future research may throw some light on other factors not as yet appreciated, that will explain this lack of relation.

[1] 1924 Book of A.S.T.M. Standards.

The copper-clad steel wires were examined in cross-section to note the distribution of the copper covering over the steel, with the results shown in Fig. 4. The wires were embedded in a matrix of low-melting-point alloy for convenience in developing the cross-sections.

DESCRIPTION OF TEST PROCEDURE AND RESULTS

The corrosion box was placed in regular operation on July 23, 1923, and continued without interruption except for Sundays, holidays and periodic examinations of the specimens, until June 30, 1924, at the termination of 793 cycles. In actual equivalent time this was equal to $264\frac{1}{3}$ days of 24 hours each, or 0.725 year of continuous exposure in the box. The test was stopped at this point because one or more definite failures had developed in the coatings of all of the galvanized wires, although varying widely in severity and superficial area. It was therefore concluded that sufficient data were in hand to warrant conclusions on the main point of the original objective.

Detailed Log.—An attendant was always on duty in the vicinity of the corrosion box during the test, and manipulated the necessary changes in test conditions at the proper times. A running log of all such changes was kept throughout the test, noting the time of each change, box temperatures, and other pertinent information.

Periodic Examinations.—From time to time, at roughly 100-cycle intervals, the test was stopped, the box opened and the specimens removed for careful examination to detect evidences of corrosion. In most instances, short pieces (about 3 in. long) were removed from the ends of the 14-ft. specimens and coated with a transparent (celluloid) solution. These short specimens were kept as permanent exhibits to show the progressive behavior of each main specimen throughout the test. Whenever the specimens were replaced in the box after such examination, care was taken to reverse them, end for end, so that any differences in corrosive effects throughout the length of the box would tend to be equalized.

These periodic examinations of the specimens were conducted by an experienced chemist, who made notes of the appearance and condition of each specimen at each examination, so that a careful record was preserved, in addition to the exhibits before mentioned.

Smoke and Temperature Conditions.—Chemical analysis of the coal-smoke atmosphere in the box on two different occasions showed that the carbon dioxide ranged from 5.1 to 8.6 per cent, tending toward the lower figure at the far end, or exit point of the gases. Sulfur determinations made at the center of the box gave 86 parts, by volume, of sulfuric acid per million parts, by volume, of total gas.

Temperatures in the box ranged from approximately 14 to 42° C. (57 to 108° F.) under different conditions, tending toward the upper figure at the end of the humid air period and toward the lower at the end of the spray period.

TABLE V.—SUMMARY OF RESULTS OF CORROSION TESTS ON ELECTRICAL CONDUCTORS.

Identification Mark	Wire or Strand	Bare or Coated	Description	Conditions Noted at Ends of Various Cycles								
				99	207	302	359	416	461	573	686	793
1-A	Wire	Bare	Aluminum	S. C.	....	S. C.	½ C.	....	....	....	C.	C.
1-B	Strand	"	Steel-Reinforced Aluminum	....	....	S. C.	S. C.	....	....	....	C.	C.
2-A	Wire	Bare	Copper	S. C.	....	S. C.	C.	....	C.	....	C.	C.
2-B	Strand	"	"	S. C.	S. C.	....	....	....	....	....	C.	C.
3-A	Wire	Bare	Copper-Clad Steel	S. C.	....	C.	C.	C.	....	....	C.	C.
3-B	Strand	"	" " "	S. C.	S. C.	S. C.	....	½ C.	C.	....	C.	C.
4-A	Strand	Double-Galvanized	Extra High Strength Steel[a]	....	....	....	S. C.	1 F.	....	....	4 F.	⅓ F.
4-B	"	" "	Siemens-Martin Steel[a]	....	....	....	S. C.	....	....	....	....	½ F.
4-C	"	" "	High-Strength Steel[a]	....	....	....	S. C.	....	....	....	3 F.	½ F.
4-D	"	" "	" " " [b]	....	....	....	S. C.	....	....	....	12 F.	⅔ F.
5-A	Wire	Bare	Ingot Iron	C.	....	C.	D.P.	....	....	[c]	....	....
5-B	"	Double-Galvanized	" "	....	C.	½ C.	½ F.	⅔ F.	....	¾ F.	....	9/10 F
6-A	Wire	Double-Galvanized	Best Best Iron	....	....	....	1 F.	....	2 F.	¾ F.	All F.	All F.
6-B	"	" "	Steel	....	....	....	....	5 F.	....	⅘ F.	9/10 F.	All F.
6-C	"	" "	Extra Best Best Iron	....	S. C.	....	½ F.	½ F.	½ F.	⅔ F.	9/10 F.	All F.
6-D	"	" "	Swedish Iron	....	....	....	....	....	....	¼ F.	⅓ F.	⅘ F.
6-E	Strand	Double-Galvanized	Siemens-Martin Steel	....	....	S. C.	....	....	....	3 F.	⅓ F.	¾ F.
6-F	"	" "	Steel	....	....	S. C.	S. C.	C.	....	5 F.	....	All F
6-G	"	" "	High-Strength Steel	....	....	....	S. C.	....	....	....	3 F.	½ F.
6-H	"	" "	Extra High Strength Steel	....	....	S. C.	....	....	....	....	3 F.	....
6-I	Wire	Bare	Swedish Iron	C.	C.	C.	D.P.	....	....	....	....	....

[a] One tinned hard-drawn copper wire in the center of the strand.

[b] One galvanized copper wire in the center of the strand.

[c] denotes the advance of corrosion to a point where the specimen is reduced to one-half of its original diameter.

S. C. denotes slightly corroded.

C. denotes badly corroded.

F. denotes local point of failure of protective coating.

D. P. denotes deeply pitted.

Integers denote number of separate local points of failure; thus 3 F. denotes three points of failure.

Fractions denote proportion of total length of specimen which has failed; thus ¾ F. denotes failure over three-fourths of the total length.

Summary of Results.—To present the detailed notes covering the progressive condition of all the specimens at each of the nine times of examination would require more space than is available. Consequently the results of these examinations have been condensed into very brief form and assembled in Table V. Every specimen in the

box showed at least some corrosion by the end of the test and practically all of the coated specimens developed substantial or complete failures of their coatings.

The specimens of aluminum, copper and copper-clad steel all behaved in much the same manner, remaining bright for a time after the test started, then discoloring, first in patches and then all over; at this point the surfaces exhibited definite evidence of incipient corrosion; this then developed gradually into an exceedingly thin scale, which finally became easily loosened and detachable, revealing another layer of oxide beneath. The actual loss of metal was not measured, but was unquestionably very slight. Although the surfaces were strongly attacked, there were no distinct failures.

TABLE VI.—COMPARISON OF TENSION TESTS ON CERTAIN SPECIMENS BEFORE AND AFTER SUBJECTION TO CORROSION TESTS.

Identification Mark	Material	Elapsed Cycles	Breaking Load, lb.		Elongation in 10 in., per cent	
			Before Corrosion	After Corrosion	Before Corrosion	After Corrosion
1-A	Aluminum Wire	793	577	549	3.8	1.8
2-A	Copper Wire	793	1337	1333	1.0	1.4
3-A	Copper-Clad Steel Wire	793	2030	2000	1.5	1.6
5-A	Bare Ingot Iron Wire	573	732	370	24.9	...
5-B	Galvanized Ingot Iron Wire	793	935	767	15.0	8.7
6-A	Galvanized Best Best Iron Wire	793	1072	956	17.3	13.8
6-B	Galvanized Steel Wire	793	1304	1182	15.2	12.1
6-C	Galvanized Extra Best Best Iron Wire	793	1061	821	16.3	8.6
6-D	Galvanized Swedish Iron Wire	793	1075	927	13.1	9.5
6-I	Bare Swedish Iron Wire	793	880	461	23.4	...

The specimens of galvanized iron and steel all behaved in a characteristic manner, remaining bright for a time, then discoloring in grayish patches, becoming general; the color next turned dark gray and then became blackish, but there was more tendency to develop pits than to scale; the last stage was the appearance of reddish spots of rust, which ultimately spread and enveloped the specimen as a whole. When a rust spot once develops, the succeeding action is first a slight swelling or increase of volume, which often causes splitting of the coating, and thus the area of failure tends to spread out. The characteristic blackening of the coatings in the last stages before incipient failure, is thought to be indicative that the outer portion of the coating, consisting for the most part of pure zinc, has disappeared, leaving only the layer of zinc-iron alloy as the remaining protection.

The two samples of uncoated iron wire, ingot iron 5-A, and Swedish iron 6-I, both commenced to corrode from the start of the

test and lost metal rather rapidly. The ingot iron lost metal at considerably faster rate than the Swedish iron.

Tension Tests After Corrosion.—In Table VI there is a summary of tension tests made on certain specimens after the completion of the corrosion tests, with comparative figures showing the strength of the same specimens before corrosion. It is interesting to compare the loss of weight, by corrosion, as given in Table VII for the bare iron wires, with their respective losses of strength, given in Table VI. In the case of ingot iron, the comparison appears rational, but in the case of Swedish iron it does not, perhaps because of pitting.

TABLE VII.—LOSS IN WEIGHT DURING TEST.

IDENTIFICATION MARK	ELAPSED CYCLES	PER CENT OF ORIGINAL WEIGHT	LOSS IN WEIGHT, PER CENT
5-A	461	49.4	50.6
	573	49.7	50.3
6-I	461	82.4	17.6
	573	86.0	14.0

TABLE VIII.—COMPARISON OF FAILURES ON GALVANIZED SPECIMENS.

Identification Mark	Wire or Strand	Description	Number of Cycles to Failure	Number of Dips to Final Failure	Weight of Coating, oz. per sq. ft.
4-A	Strand	Extra High Strength Steel (copper center)	793	6	1.09
4-B	Strand	Siemens-Martin Steel (copper center)	793	6	1.22
4-C	Strand	High Strength Steel (copper center)	793	5	0.86
4-D	Strand	High Strength Steel (copper center)	793	5	0.94
5-B	Wire	Ingot Iron	359	5	1.05
6-A	Wire	Best Best Iron	573	8	1.02
6-B	Wire	Steel	573	6	1.15
6-C	Wire	Extra Best Best Iron	359	7	0.82
6-D	Wire	Swedish Iron	686	7	1.28
6-E	Strand	Siemens-Martin Steel	686	6	0.97
6-F	Strand	Steel	793	5	1.15
6-G	Strand	High-Strength Steel	793	5	1.02
6-H	Strand	Extra High Strength Steel	793+	5	1.18

Comparison of Failures.—It appears reasonable to denote failure of the zinc coating as that point in the test at which the existence of iron rust becomes rather prevalent over the whole length of a specimen, rather than a few isolated small occurrences of it; especially if the isolated occurrences are of no appreciable depth or extent. Taking the condition of failure over at least one-third of the entire length as denoting the point at which a specimen as a whole has failed, from Table V, and contrasting the number of cycles up to this point with the data in Table IV, produces the interesting exhibit given in Table VIII. These results indicate that neither the number of dips (to failure) in the Preece test, nor the weight of the coating,

are sufficient of themselves to predict the durability of any particular coated specimen in these tests.

Normal Atmospheric Corrosion Test.—Specimens taken from the same original wires and strands as those in the corrosion box were suspended on an outdoor rack on the roof of the Laboratory and examined periodically. This test is still in progress and it is hoped that in time the results will permit of some sort of comparison with the results in the corrosion box, from which it will be possible to equate the box conditions into equivalent outdoor exposure for one specific locality. Thus far, up to 417 days of exposure last reported, the specimens were all in good condition, except of course 5-A and 6-I, which are bare iron, and with the exception also of one local point of failure on 3-B. None of the galvanized specimens had yet developed any evidences of exposure of the zinc-iron alloy beneath the outer

TABLE IX.—COMPARATIVE BEHAVIOR OF BARE INGOT IRON WIRE IN ACCELERATED TEST AND NORMAL ATMOSPHERIC TEST.

Accelerated Test			Normal Atmospheric Test	
Elapsed Cycles	Elapsed Days	Average Diameter of Specimen Removed, mils	Elapsed Days	Average Diameter of Specimen Removed, mils
0	0	147	0	147
99	33	149	100	152
207	69	158	180	159
302	101	129	280	153
359	120	139	350	145
461	154	113	...	...
573	191	117	...	...

layer of zinc. Comparisons of the change in diameter of specimen 5-A, in the corrosion box and on the outdoor rack, are presented in Table IX. It is interesting to note that under both exposure conditions the first effect of corrosion is to cause a swelling of the specimen, which later disappears as the corrosion advances.

CONCLUSIONS

The conclusions drawn as to the relative performance of the various specimens in resisting the attack of simulated corrosive conditions of unfavorable outdoor atmospheres, under accelerated application, are as follows:

1. The copper and aluminum wires and strands did not show serious corrosion up to 302 cycles and there were no marked failures at any time either in the copper-clad steel or in the reinforcing wire of the aluminum cable.

2. There is little, if any, difference between the corrosion in solid wire and stranded wire of the same grade of material.

3. The copper-center steel strand resists corrosion fully as well as the standard material of the same grade.

4. The higher the carbon content of the galvanized iron and steel specimens, the greater is the apparent resistance to corrosion.

The experience thus had with this accelerated corrosion box indicates that the method as a whole is probably reliable for the purpose intended and provides a means for obtaining accelerated results of a comparative nature. The results to date are insufficient to afford any basis for equating the life of a specimen in the box into probable life in outdoor service.

Acknowledgments.—Thanks are due especially to Messrs. F. M. Farmer, E. D. Doyle and P. F. Wehmer of the Electrical Testing Laboratories for their cooperation and assistance, and to the Indiana Steel and Wire Co. for permission to publish the results.

DISCUSSION

Mr. Rawdon.

MR. H. S. RAWDON.[1]—In reading over the paper, I could not help being struck by the similarity of this test with the one which we described in a paper before the Society last year.[2] The test to which I refer is the simulated atmospheric test developed by the Research Laboratory of the New Jersey Zinc Co. Sub-Committee VII, on Accelerated Tests, of Committee A-5 on Corrosion of Iron and Steel, is making extensive use of the test in its work. It consists in exposing the samples first to a moist atmosphere containing a known amount of SO_2 and CO_2. The samples are then taken out of that atmosphere, or rather, the atmosphere is removed and the samples are washed with a heavy spray of water; then, third, they are taken out of the box and dried. The whole cycle requires something like 24 hours. Then the cycle is repeated, and the number of cycles that the material will stand —we are talking about coated material—is taken as a measure of the life of that material.

It seemed to me that Mr. Fowle's test is very similar to this. The chemical action would necessarily be much the same. The chemical action of the bituminous smoke (upon the coating) would be that of CO_2 and SO_2, a weak acid, and then the mechanical action of the water and the drying action would be much the same as we described in our test last year. It is rather gratifying to the sub-committee to know that its test is receiving commercial application.

Mr. Breyer.

MR. F. G. BREYER.[3]—I think the committee Mr. Rawdon refers to stands on accelerated tests of corrosion of iron and steel where we stood about three years ago on paint coatings. The accelerated testing of paint coatings in the last five years has successively gone through the stages of being severely criticized without investigation, of being bellicosely investigated and finally of interested adoption. It has won a place for itself in the laboratories of many of our prominent paint companies and we will shortly have a meeting of the various laboratory representatives who have or who are about to install accelerated weathering tanks for paints and varnish.

There is just one thing to bear in mind, which I think, perhaps, was the meat of the discussion when this subject of accelerated test for metals was brought up a year ago, and that is the fact that one cannot immediately interpret these results in terms of service. If one

[1] Physicist (Metallography), U. S. Bureau of Standards.

[2] H. S. Rawdon, A. I. Krynitsky and W. H. Finkeldey, "Types of Apparatus Used in Testing the Corrodibility of Metals," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 717 (1924).

[3] Chief, Research Division, New Jersey Zinc Co., Palmerton, Pa.

Mr. Breyer. has never had an accelerated corrosion tank, and if one has not observed the corrosion of galvanized or lead coatings in service, and has had no experience in such work one cannot immediately take an accelerated corrosion test and derive any results from it. However, if the man who has been studying galvanized coatings or the man who has been studying paint and lacquer films, who is already an expert in this field, adds this one more very valuable tool to those he is already familiar with, he can immediately put himself in a position to acquire information impossible to obtain otherwise. Those who are going to do accelerated corrosion work will do so not because it is going to tell them the whole story but because they believe it is one more good tool in the hands of an expert.

Mr. McKay. MR. R. J. MCKAY.[1]—One of the most interesting angles of these tests is the attempt which will be made to check them with actual atmospheric corrosion. After all, this is the final criterion, and, if a proper check can be made, such results are more important than any simple comparisons. The true mechanisms of atmospheric and other corrosions are much better known than they were a few years ago, and therefore it should now be possible to show fairly good agreement.

I should like to ask questions on the following points in the conclusions of Mr. Fowle's paper:

"There is little, if any, difference between the corrosion in solid and stranded wire of the same grade of material." Does this refer to difference in the rate of corrosion of the whole stranded wire, or in the rate per unit of surface? The stranded wire has a larger total surface, of course, than a single wire, and this question is therefore important.

In conclusion No. 3, "The copper-center steel strand resists corrosion as well as the standard material of the same grade," the author does not go so far as to say that the copper-center strand resists corrosion any better than the standard material. I wonder if it is not better. Recent work has shown the reason why a pit, once having started in the corrosion of steel, continues, regardless of the structure of the steel. This cause of the continuation of pits will also act on the inside layers of stranded wire, causing more rapid corrosion toward the inside than on the most exposed portions. This rapidly-corroded inside is anodic, and a copper center might be expected to decrease the tendency to be anodic and make such wire better than wire made from one material. This would be, of course, contrary to our usual unreasoning fear of electrolytic effects between two metals subject to corrosion.

Mr. Breyer. MR. BREYER.—Mr. McKay's remarks occasion me to repeat that in the past I think there has been too much tendency to ask an acceler-

[1] Superintendent, Technical Service, International Nickel Co., 67 Wall St., New York City.

ated test to check with a so-called actual service test. Now, the most important thing that a study of accelerated tests brings out is that there is no such thing as a standard outdoor exposure of any sort. We are going to get different results at Sandy Hook than we do at Key West, and we are going to get different results over at Pennsylvania State College than we do at Altoona or Pittsburgh. Do not be disappointed because the accelerated test does not check your particular exposure, or panels exposed in different parts of the country having widely different weather conditions. You are not going to get anywhere if you try to standardize those weather conditions; the only place I know of where you can standardize them is in the laboratory and that is one of the biggest arguments for accelerated weathering equipment. It can be standardized. Mr. Breyer.

Incidentally, one of the most helpful things we ever did was to establish a cooperative Government weather bureau. That weather station of ours has paid for itself a dozen times over. We now know something, at least, about the weather cycles in our neighborhood.

MR. F. F. FOWLE.—Answering the question on conclusion No. 2, "There is little, if any, difference between the corrosion in solid wire and stranded wire of the same grade and material," that was a general observation made on those samples of wire and the strand which contained the same material or very closely the same material. It was an observation based on the external appearance and the external examination. On conclusion No. 3, "The copper-center steel strand resists corrosion fully as well as the standard material of the same grade," I would say there was no indication that it withstood it better but that the conclusion here given was felt to be warranted by the appearance of the samples themselves. Mr. Fowle.

On the point made about different atmospheres about the country, it seemed to me in reflecting on this whole subject that there might be a possibility of doing some research work or making some study of the atmospheres about the country with an effort, perhaps, not to chart them exactly but to some extent classify them and to make some physical and chemical tests over a representative period of time, which would tend to give us some more information as to what the cycles are of temperature and of rainfall and moisture, and in particular what impurities of a corrosive nature are present in these atmospheres. It is natural enough that they will vary; they will vary from time to time during the same day; they will vary from location to location, but taken over a long period of time the law of averages ought to eliminate some of those variations, and it would seem as though a study of that sort might be helpful in this whole problem and be worth the while.

THE EVALUATION OF CORROSION TESTS

By E. Blough[1]

Synopsis

The paper, after brief mention of the several methods commonly used to determine the effect of corrosion of metals, proposes as worthy of further investigation a method that has been used by others, namely, comparing the physical properties of the metal before and after corrosion. Tests are described in which specimens of five commercial non-ferrous metals, in the form of tension test specimens, were subjected to corrosion in a salt spray and tested for tensile strength and elongation after being subjected to salt spray for various periods of time. This method has inherently the advantage of disclosing the effect of corrosion upon the residual metal which is apparently unattacked, and it is believed that the method should be a valuable adjunct to other methods for evaluating corrosion tests.

One of the most serious problems confronting investigators to-day is the question of corrosion of metals. Among the puzzling questions connected with the testing of corrosion is the determination of the effects of corrosion. It is not the intention, in this paper, to consider any of the various methods of producing corrosion, nor is it the intention to consider the relative merits of metals under conditions of test or under conditions of use. It is desired, however, to bring to the attention of those interested in the subject, a method of determining the effect of corrosion on metals, a method that has been used by others,[2,3] but which seems worthy of further consideration by those investigating this subject.

There have been proposed, and used, several methods of determining the effect of corrosion. While there will be no attempt to discuss all the methods that have been used, it would seem desirable to enumerate some of those which have been most prominently considered and probably utilized most commonly.

The most simple method, when no attempt is made to acquire data, is inspection, and comparing the corroded materials either with uncorroded material or with other materials used for a similar purpose.

[1] Technical Director, Aluminum Co. of America, Pittsburgh, Pa.

[2] P. D. Merica, "The Embrittling Action of Sodium Hydroxide on Mild Steel and Its Possible Relation to Seam Failures of Boiler Plate," *Chemical and Metallurgical Engineering*, Vol. 16, p. 496, (1917).

[3] D. Basch and M. F. Sayre, "Resistance of Various Aluminum Alloys to Salt-Water Corrosion," *Mechanical Engineering*, Vol. 46, p. 199 (1924).

Such a method is open to many objections and can serve only a temporary use, as it leaves no record by which one investigator can compare his work with that of another.

Another method frequently used is that of weighing the test sample before and after its having been subjected to the corrosion test. While this yields specific data its disadvantage lies in the difficulty of carrying out the test satisfactorily. Frequently, there are adherent products of corrosion which cannot be removed from the sample without at the same time removing some of the residual uncorroded material. Therefore, this method of determining the effect of corrosion can be used only where the opportunity for such an error is non-existent.

A third method is to determine analytically the products produced by the corrosion test. This again has the advantage of yielding

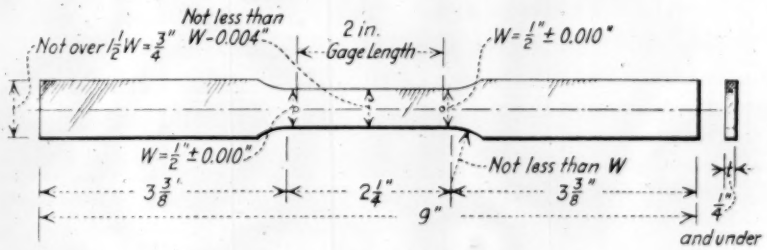


FIG. 1.—Test Specimens.

data that can be recorded, but can be used only when one is certain that all the products produced by the corrosion are available for analytical determination.

Still another method that is sometimes used is to measure the depth of the corrosion. This is carried out by attempting to measure the depth to which corrosion has penetrated, or by removing the surface until the greatest depth of the action has been determined. This procedure will also yield data, but those who have had occasion to make corrosion tests realize how difficult it is either to measure the depth of a corrosion pit, or to remove satisfactorily the surface by scraping, or otherwise determining the depth to which corrosion has progressed.

One common criticism applies to all of the above methods of determining the effect of corrosion, and that is they do not disclose the effect corrosion has produced upon the residual metal which is apparently unattacked. Neither do they disclose a condition wherein there may be no apparent corrosion, but where there may be an interior disintegration.

One method of measurement which avoids the above-mentioned criticism is to subject the specimens after corrosion to physical tests. If the corrosion specimens are prepared in the form of test pieces, then they may be tested at least for tensile strength and elongation. By measurement of these properties both before and after the material has been subjected to corrosion, and for various lengths of time, a very good picture is obtained as to the effect of the corrosion on the material itself.

In order to demonstrate the use of this method, the author conducted a series of salt-spray tests on five commercial non-ferrous

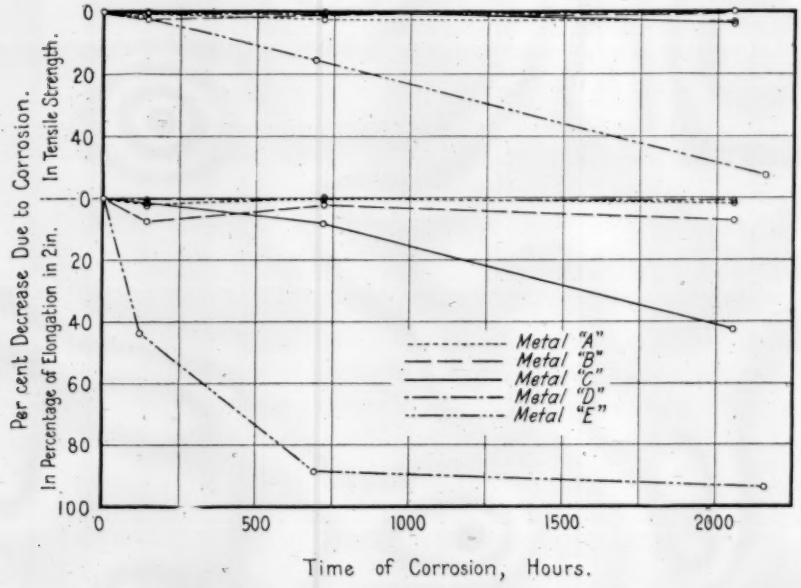


FIG. 2.—Effect of Corrosion on Physical Properties.

metals. In this paper the metals are listed as A, B, C, D and E, because it is desired only to show the results to be obtained by the use of physical tests as a means of evaluating corrosion, and it is not desired to present these data for the purpose of comparing the merits of the metals used for tests. It seems desirable moreover to avoid this comparison because, since the samples were secured by purchase from warehouse stock, they might not be representative of the products which the manufacturers would normally select as most resistant to an adverse condition. The materials were all in sheet form and the samples used for corrosion test were standard sheet test specimens as shown in Fig. 1. The materials were tested before being subjected

to the salt-spray test, and blank samples were also held for test after the completion of the experiment in order to be sure that no aging effect had taken place in any of the materials, due to lapse of time.

Three series of samples were used in the test; the first for a period of 120 to 144 hours; the second 689 to 720 hours, and the third series was removed at the end of 2056 to 2160 hours. The samples, on removal from the salt-spray tank, were immediately washed and dried in order to avoid any continuation of corrosion due to exposure to the air, and then subjected to tensile strength and elongation measurements.

After the data were accumulated from the complete series, the results were calculated on the basis of percentage of loss using the uncorroded material as the unit basis. The results obtained from this tabulation are shown in Table I. These results are also shown in

TABLE I.—EFFECT OF CORROSION ON PHYSICAL PROPERTIES.

Sample Mark	Change in Tensile Strength, per cent	Change in Elongation, 2 in.	Change in Tensile Strength, per cent	Change in Elongation, per cent	Change in Tensile Strength, per cent	Change in Elongation, per cent
	144 Hours		720 Hours		2056 Hours	
A	−0.79	−2.09	−2.47	+1.12	−3.33	−1.93
B	−2.37	−7.61	−0.96	−2.17	+0.14	−7.61
C	+0.14	−1.78	+0.21	−8.28	−3.79	−42.3
D	−0.62	−0.44	−0.40	−0.44	+3.68	−1.11
	120 Hours		689.5 Hours		2160 Hours	
E	−1.32	−43.6	−15.5	−88.7	−52.9	−93.8

the curves of Fig. 2, each value representing the average of tests on three specimens.

It will be seen from the study of these results that metal E shows a gradual decrease in tensile strength with a very rapid loss in elongation. It may also be noted that metals A and D under this test show relatively little attack, while metal C shows a loss of elongation although its tensile strength has been but little affected.

There is one condition wherein this method of testing might not yield an indication as to the effects of corrosion. That condition is where the corrosion takes the form of widely scattered pits under which circumstances test samples might be subjected to corrosion and no pitting occur on the samples selected. When pitting of such a character does occur, it is generally of most importance to those using the material as containers. By increasing the number of samples tested under such circumstances it is probable that physical test specimens will show the effect of such a type of corrosion.

It is evident that if tests of a similar character were conducted on various metals under different corroding influences, valuable information would be obtained as to the influence of corrosion upon the properties of these metals. The rate of the progress of corrosion under different conditions and lengths of time would be shown. Such a method should be a valuable adjunct to other methods for evaluating corrosion tests.

DISCUSSION

MR. E. E. THUM.[1]—Mr. Blough's paper is worthy of commendation on several counts; chief among them is its conciseness. However, I believe it might be extended to give the range of values observed in the tension tests. The results are given to a fraction of 1 per cent, and the presumption is that the corroded samples check pretty closely, but I hope that the individual test results will be printed. Mr. Thum.

Something might be said about the entire question of the interpretation of accelerated tests. Doubtless it has occurred to several of you frequently that fatigue testing and corrosion testing have a good deal in common. In the first place, there is no acceptable theory explaining why or where corrosion or fatigue starts, or why it continues after it is once started. Having no acceptable theory of the mechanism, the engineer must rely upon long-time tests of both fatigue and corrosion in order to make up his own mind which materials shall suit his purpose best. In the second place, we have no accelerated test, generally accepted, for either corrosion or fatigue, and I think any accelerated test for either, must check up with the available long-time tests very closely indeed, to bear much weight with the engineer. That, of course, is simply because we have no precise knowledge of the mechanism of the action.

It seems to me that the results of the work of Committee A-5 on Corrosion of Iron and Steel—long time exposures of sheet metal to atmospheric test—could be used to very great advantage for the proper evaluation of accelerated corrosion tests, at least those which are designed to simulate the action of atmospheric corrosion. For instance, would it not be possible to take such a test as the one which Mr. Blough has described and check it back against the very materials which were used by Committee A-5? Suppose that pieces of these very sheets were exposed for six months or a year, a moderate length of time, and then tested according to Mr. Blough's criterion. Now if this suggested test is able to locate the various sheets in the order in which they actually failed in the exposure tests, and if I remember rightly the order of failure was quite consistent, then the engineer would be willing to say that we have at least found an accelerated test for atmospheric corrosion. Then some time or other in the future when some metallurgist or manufacturer bobs up with a new

[1] Manager, Publicity Department, Linde Air Products Co., New York City.

Mr. Thum. sheet which is "guaranteed to beat the world," we could find out in six months something about it, and have a result which would be fairly convincing, and which might reasonably be expected to be borne out over the course of 10 or 15 years' actual use.

Mr. MacKenzie. MR. J. T. MACKENZIE.[1]—I should like to ask if the tension tests on the corroded specimens are based on the original area or are they calculated from the final area?

Mr. Blough. MR. E. BLOUGH.—I can answer that right away, on the original area. Every specimen was measured carefully before it was put into the salt spray test. All measuring and marking was done beforehand.

Mr. Job. MR. ROBERT JOB.[2]—I should like to ask Mr. Blough whether the very low elongation on sample E is roughly a measure of the extent of pitting. In other words, was sample E very badly pitted, very much worse in that respect than the others, and can we get a relative idea as to the amount of pitting by the difference shown on the curve, or is that due to some other cause?

Mr. Blough. MR. BLOUGH.—What we term corrosion on that particular metal did not take the form of pitting. It was really more of a general loss. In fact, there are very few cases where it took the form of actual pitting.

Mr. Speller. MR. F. N. SPELLER.[3]—The interesting experiments described by Mr. Blough indicate again, that with certain metals it is necessary to have a measure of the disintegration going on below the surface under corrosion as well as a measure of the maximum depth of pitting and the average loss in weight per unit of area. That is to say, the extent of corrosion should be expressed in at least three different ways to tell the whole story: (1) average loss per unit of area, (2) maximum depth of pitting, (3) internal disintegration. We have indicated the tendency to pitting by a figure obtained by dividing the maximum depth of penetration in inches per year by the average penetration in inches per year which has been termed the "pitting index" or "pitting factor." But with some metals it is impractical to clean out all the disintegrated or corroded material and in such cases the depth to which the corrosion has penetrated and injured the metal can only be determined by a measure of the physical properties or by microscopic examination. In this connection it might not be out of place to call attention to the variety of units which are used to express the rate of corrosion and the desirability of standardizing these units so that the results of different investigators will be comparable. The necessity for harmony in the

[1] Chief Chemist, American Cast Iron Pipe Co., Birmingham, Ala.
[2] Vice-President, Milton Hersey Co., Ltd., Montreal, Canada.
[3] Metallurgical Engineer, National Tube Co., Pittsburgh.

use of such units was recently called to the attention of the U. S. Bureau of Standards and we understand that the matter is now receiving consideration at their hands. **Mr. Speller.**

MR. H. S. RAWDON.[1]—The Bureau has drawn up a recommendation on this subject and it was submitted to Mr. Corse of the National Research Council's Corrosion Committee. I do not know that you would call it standardizing, but we have made certain definite recommendations. In general I think, from the utilitarian standpoint, a man is interested in the corrosion question as to how much the usual life of his product may be shortened, so that it narrows down to a question of either how much of the metal is removed from the surface, or how deeply it is pitted. In some cases the depth of pitting would be the deciding factor, so that any method of presenting corrosion results, at least according to the Bureau's recommendation, should take cognizance of those two facts. We recommend that the usual loss of weight per unit area per unit time be given but that the result be translated into terms of depth of penetration, that is, *average penetration*, over the whole surface. This calculation is simple enough by making use of the density of the material. In addition to that, if you have pits the *maximum penetration* should be recorded, and a useful number or "index" indicating the behavior of any particular material under any particular test is to take the ratio of these two, the maximum to the average penetration, that of which Mr. Speller spoke. We have also recommended that that be given. In addition, there are cases, as Mr. Blough has presented, where there is no pitting but a decided loss of ductility and some loss of tensile strength. He has just shown that the ductility may drop almost to zero. We have had similar cases. In that case the loss of weight does not mean a thing in a practical way and you have no pitting to indicate that the material is deteriorating, although there is a real deterioration of the material going on, we feel sure. It is no longer the original material. In those cases we have recommended that where you can get suitable specimens the tension test should be used. In that way you get a measure, but in a different way, of the depth of penetration of the corrosive action. In brief, I think that covers the recommendations that the Bureau made regarding the method of presentation of the results of corrosion tests. **Mr. Rawdon.**

MR. W. M. PIERCE.[2]—I have some data on a point raised twice in this discussion, namely, whether the loss of strength is a direct indication of the percentage reduction of area. We have used loss of **Mr. Pierce.**

[1] Physicist (Metallography), U. S. Bureau of Standards.

[2] New Jersey Zinc Co., Palmerton, Pa.

Mr. Pierce. tensile strength very extensively as a measure of corrosion in a certain class of work. In some cases we have used it on specimens where we could determine microscopically that the penetration had been very uniform to a slight depth and yet the loss in tensile strength was out of all proportion to the penetration. The explanation for this is probably the very irregular cross-section, or rather the very rough cross-section that is left, giving the effect of a notched bar in the tension test.

I believe that this is a point which should be kept in mind before we try to convert directly from loss in tensile strength to reduction of area. The loss in tensile strength is very important, but I do not believe that a simple conversion between it and reduction in uncorroded cross-section is possible.

Mr. Job. MR. JOB.—There was an instance we had a year or two ago that is rather interesting, I think, and gives an idea of the care which should be taken in forming any definite conclusion as to the cause of corrosion. There was a boiler tube in a marine boiler which failed after about six weeks' service due to perforation of several of the tubes. The trouble was that a number of holes about $\frac{1}{4}$ in. in diameter formed, all within a distance of about 2 ft. Each had penetrated completely through the tube. The rest of the tube was in practically perfect condition with the mill scale still adhering. In that case we investigated and found that the whole trouble was due to the fact that hot air had been forced from the injector against a particular portion of the tube, so that the heat and the moisture and the oxygen caused the very localized corrosion. In that case the actual loss of the metal in proportion to the total weight was practically nothing, but at the same time the tube was completely ruined and had to be removed from the boiler after service of only 6 weeks, involving considerable loss and delay.

Mr. Breyer. MR. F. G. BREYER.[1]—I think Mr. Blough's point is very well taken, that a good deal of the corrosion data in the past has been almost entirely chemical. We do not care particularly whether a zinc battery can disintegrate ten or twenty grams per centimeter; what we are interested in is whether it went through in one spot or not, even if only a gram was dissolved off. I think it is perhaps a little premature to give these corrosion units that Mr. Speller has suggested. There is too much of a tendency to try to report these things as standard. It strikes me that the most hopeful thing we can look forward to in connection with corrosion problems is the object lesson given us in the use of the microscope by the surgeon, the biologist, and the metal-

[1] Chief, Research Division, New Jersey Zinc Co., Palmerton, Pa.

lographist. The greatest advance in the study of metals was made when we were able to micro-section them and see their actual physical structure. We have had chemical attack and the corrosion problem for a long time. Our chemical story is miles ahead of our physical story. Let us go back and use the microscope a little more. I do not see why we cannot make micro-sections of metals and study under the microscope the actual effect of various corroding agents. **Mr. Breyer.**

In the case mentioned by Mr. Peirce, in which we are actually applying Mr. Blough's criterion, namely, loss of physical strength of material after corrosion, the microscope shows just how corrosion has proceeded. Furthermore, our use of the microscope to study the corrosion and disintegration of certain zinc base alloys has given us more information, I am sure, than the same amount of time spent studying the chemical reactions involved. If I gather what Mr. Blough has in mind, it is just that very point; a little more physical diagnosis, a little more of what we can see and build up from the structural standpoint is what we want rather than chemical evidence.

MR. M. F. SAYRE[1] (*by letter*).—Speaking from considerable experience with the method, the writer would like to second Mr. Blough's remarks regarding the desirability of strength tests as a means of obtaining definite quantitative values of rate of corrosion. I have been using this method for the past five years, and would strongly recommend it for use provided (1) the importance of the question justifies making up the tension test specimens in duplicate and (2) the rate of corrosion is great enough to give a reasonable loss of strength in the time available. For materials with a very slow rate of corrosion, as, for instance, stainless steels or bronzes in salt spray, either appearance or change in weight will form a more sensitive means of comparison. **Mr. Sayre.**

Comparative results, using this method, and checking as against other methods, on quite a number of aluminum alloys after eight weeks in salt spray at 40° C. are given in the accompanying Table I.[2] The losses in strength vary from 1 per cent up to 35 per cent, a wide enough range to make the figures definitely mean something. The "per cent loss in area to bottom of pits" was obtained by imagining that over the 2-in. gage length, the metal was ineffective to a depth equal to the maximum depth of pits, and computing the resultant percentage loss of area. The close agreement between this column and the percentage-loss-in-strength column coupled with the fact that Brinell and sclero-

[1] Associate Professor of Applied Mechanics, Union College, Schenectady, N. Y.

[2] For more complete information concerning these tests, see article on "Resistance of Various Aluminum Alloys to Salt-Water Corrosion," by D. Basch and M. F. Sayre, *Mechanical Engineering*, April, 1924, p. 199.

Mr. Sayre. scope readings were virtually the same for both air and salt spray, indicated that the loss in strength was a surface action (due to loss in area and to roughening of the surface), and was not caused by internal disintegration of the material. There is a peculiar exception in alloys *Q* and *W*, particularly the first one. These alloys are susceptible to

TABLE I.—COMPARATIVE CORROSION DATA ON ALUMINUM ALLOYS.

Alloy	Loss in Strength, per cent	Loss in Area to Bottom of Pits, per cent	Relative Rating Based on				Elongation in 2 in., per cent	
			Appearance	Gain in Weight	Depth of Corrosion	Percentage Loss of Strength	Air	Salt Spray
INITIAL SERIES								
A	1.1	1.2	2	2	3	2	15.6	16.6
B	1.0	1.3	3	1	2	1	26.2	26.3
C	4.7	6.4	7	7	8	7	18.4	16.2
D	2.8	2.1	5	5	4	5	21.6	20.0
E	3.6	4.2	6	6	7	6	16.2	13.8
F	1.5	0.8	1	4	1	3	24.0	22.8
G	20.2	15.3	14	16	14	14	1.3	1.7
H	17.5	17.4	13	15	13	12	1.7	1.7
I	11.4	12.5	10	12	12	10	3.6	3.1
J	15.0	10.3	8	9	11	11	7.0	6.4
K	2.6	3.3	4	3	5	4	12.6	10.4
L	35.3	32.1	15	14	16	16	0.7	0.5
M	26.1	27.2	16	13	15	15	0.6	0.6
N	4.7	5.5	9	8	6	8	3.6	4.8
O	10.3	11.8	11	10	9	9	0.4	...
P	18.3	12.4	12	11	10	13	0.4	0.4
SECOND SERIES								
Q	[a]	6.8	8.5	9.5	8	...	5.2	5.0
R	14.2	6.4	12	14.5	8.5	11	2.2	1.8
S	8.3	4.7	12	13.5	8	9	4.6	5.5
T	10.9	8.0	12	13.5	8	9.5	10.0	7.5
U	7.2	9.5	6.5	9	8	8.5	5.4	3.5
V	1.3	1.1	4.3	5.5	4	2.5	4.6	5.2
W	[b]	0.3	1	1	2	1	5.2	6.0
X	4.4	1.0	7	7	3.5	6.5	7.6	7.6
Y	9.0	8.3	9.5	10.5	7.5	8.5	0.2	0.9
Z	8.0	7.2	9.3	9	7.5	8.5	4.5	4.3
AA	3.5	9.6	7	7.5	8	6	0.2	0.4

[a] Gain 11.0 per cent.
[b] Gain 3.9 per cent.

heat treatment and to aging, and apparently the slight difference in temperature (40° C. in salt spray, 25° C. outside) over a period of eight weeks, was sufficient to cause an appreciable increase in strength in salt spray. This is verified by the fact that the Brinell hardness also had increased.

Surface corrosion evidently has not greatly affected the percentage elongation. This was to be expected. The roughening of the surface would cause some decrease in elongation, but simple change in area

of cross-section should not. Internal disintegration naturally would have a different effect. Mr. Sayre.

I would like to call attention to the use of gain in weight rather than loss in weight as a criterion. In the salt spray box, the oxide coatings formed on most materials are well nigh impossible to remove, and loss in weight becomes virtually useless. As an alternative, we have gone to the other extreme, handled the specimens as carefully as possible to avoid jarring off the incrustations formed, and then measured the gain in weight. This gain in weight, of course, is a composite figure due to (1) difference in weight between the original metal and the oxides or hydroxides, etc., formed from it; plus (2) weight of salt deposit formed on the specimen; less (3) weight of corrosion products removed either in solution or by rough handling. The method is feasible only (*a*) where the products of corrosion are relatively insoluble in the solution used, (*b*) where the solution is kept dilute enough so that the salt does not crystallize out in quantities on the samples (4-per-cent strength salt solution was used for these tests, rather than the 20-per-cent solution often mentioned), and (*c*) up to that point where the incrustation becomes thick enough to begin to drop off. The gain in weight used here was taken at the end of a period of two weeks only, not after the full eight weeks, for this reason.

Of the different methods, judging by appearance of course takes the least time, but for obvious reasons is not entirely satisfactory. Measuring by gain in weight takes but little time, and shortens up the length of run, as reasonably indicative figures may be obtained in one or two weeks, rather than eight weeks, but calls for very careful handling, and may be used only with certain materials and certain solutions. There is also always left a residuum of uncertainty as to how much of the incrustation had been lost and what percentage of impurities was present in the incrustation as weighed. The depth of pitting is a vitally important factor in determining the life of the material, and the results as measured are easily converted into directly useful terms, and from this viewpoint this is the best method of all. On the other hand, in the salt spray and similar tests, the pits are likely to be filled with adherent deposit, so that their depth cannot possibly be directly measured, and the only method left is to measure and then carefully and slowly file down the metal over a considerable area until the bottom of the deepest pits have been reached. This is an extremely fussy, time-consuming operation, and often tremendously dependent upon judgment in knowing when to stop. Tension tests are undesirable in that loss is a composite—partly surface corrosion and partly interior change—but they are the only methods which really

Mr. Sayre. take into account this second important factor. They take a certain amount of time to perform but probably no more than is really needed to measure depth of pitting, and the results once obtained are definite and relatively independent of personal judgment.[1]

A relatively large source of possible error, particularly with cast materials, lies in the inherent variation in strength between successive test bars. This may be great enough to mask the effect of the corrosion. To reduce this to a minimum, (1) samples should be made up in pairs, a control bar for each corrosion bar, cut out adjacent to each other if from sheet, or cast in pairs in the same test bar mold to obtain the same composition and same pouring temperature and same conditions of cooling, if of cast material; (2) a sufficient number of pairs of samples should be made up to allow of averaging results, and to allow of identifying and discarding the occasional obviously erratic results obtained; and (3) air and corrosion samples should be kept under the same conditions, except for the corroding medium (that is, at the same temperature), and pulled approximately at the same time.

As other speakers have brought out, the accelerated corrosion tests are essentially comparative in value, and will tell which of two materials will probably last better under given conditions, not how long any one will last. For this reason, some one or two materials known to have a satisfactory length of life should preferably be introduced in any series of tests. Much of the criticism raised against accelerated tests has been due to neglect of that precaution. Other criticism has been due to misapplication of the results. A "salt spray" test attempts to tell how a material will behave in salt spray, on shipboard or on the seacoast and does not necessarily tell much about its serviceability under, let us say, Pittsburgh conditions. A different set of corroding agents is in action, and behavior may well be entirely different. SO_2, CO_2, organic acids, various mineral salts, even in minute quantities, each have their specific action. Stray electrical currents and electrolysis due to contact with other metals further complicate conditions. Much caution is therefore needed in interpreting the results and long term service tests under actual conditions of use will also be needed, certainly until we are better able to identify what specific conditions to guard against for any one type of use.

[1] The numerical values of percentage loss obtained, of course, depend upon the size of test specimen used, and judgment must be used in comparing results in case the size and shape was not kept the same throughout.

REQUIREMENTS OF CEMENT FOR MODERN HIGHWAY CONSTRUCTION

By A. T. Goldbeck[1]

Synopsis

This paper discusses the service requirements of concrete in concrete pavements giving the most destructive influences as those which produce tensile stresses in the pavement as follows:

1. Shrinkage of concrete due to drying of moisture.
2. Contraction of concrete due to decrease of temperature.
3. Warping of the slab due to variations in temperature and moisture.
4. Bending due to heavy wheel loads applied frequently.
5. Bending due to non-uniform subgrade support.
6. Excessive compressive stress due to rise in temperature or increase in moisture content or a combination of both.
7. Frost action.

The conclusion is drawn that concrete for highway construction would be even more suitable from a technical standpoint than it is at present:

1. If it developed high tensile strength within a very few days.
2. If it had a very low shrinkage factor under the action of moisture and temperature.
3. If it were even more durable in its long-time resistance to alternations of freezing and thawing, wetting and drying, changes in temperature and repeated loads.

The paper discusses further the various properties of cement that might be given further consideration and the attitude of various state highway testing engineers on the necessity of changes in the current specifications.

The use of portland cement for pavement construction has developed with great rapidity in the last fifteen years. Thus, we are told, "In the five-year period, 1909 to 1913, which comprises the early days of concrete roads, less than 1.5 per cent of the cement produced in the United States was used for pavements. In the five-year period, 1920 to 1924, this was increased to nearly 20 per cent. At the present time, 25 per cent of the cement made is being used in pavements of various classes."[2]

As an example of the large mileage of concrete pavements constructed in this country, it might be stated that the Federal govern-

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[2] The "Contribution of Scientific Research to the Development of the Portland Cement Industry in the United States," by D. A. Abrams, Professor in Charge of Laboratory, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

ment has cooperated with the various state highway departments in the construction of over 10,000 miles of concrete pavements, and it is estimated that 34,000 miles of concrete pavements have been built in the United States by the various municipalities, county, state, Federal government and other highway constructing agencies. Many miles of pavement surfaces remain to be constructed of which a high percentage will undoubtedly be built of portland-cement concrete. Such construction involves the expenditure of hundreds of millions of dollars for materials alone, and a considerable portion of these funds will be used for the purchase of portland cement.

In the light of these figures it is important that we know whether or not the present standard specifications for portland cement are written in a manner such that the most economical and, in general, the most satisfactory concrete pavements will be built. Is it possible that the standard requirements for portland cement should be altered in some respects to produce a cement more suitable for pavement construction than the portland cement now available under our present specifications? Is it possible that the particular demands made on concrete pavements render it advisable to use a particular quality of cement having properties not possessed by our present portland cement and, if so, is it likely that economy will result from the use of this special cement? These are questions which many highway engineers have been asking themselves because of their responsibility as public servants of obtaining the most economical highway service for the huge sums which they are expending.

SERVICE REQUIREMENTS OF CONCRETE IN CONCRETE PAVEMENTS

That we might have a better understanding of the qualifications of portland cement for pavement construction it will be well to review very briefly the various influences to which concrete pavements are subjected, tending toward their gradual deterioration, for too frequently the fact has been overlooked that concrete highways are subjected to many forces other than traffic loads.

If the history of the concrete in a pavement be followed through from the very beginning, some understanding will be had of the properties which road concrete should possess. When the wet concrete is placed on the subgrade, unless the proper precautions have been taken to insure against rapid absorption of water by the subgrade or against rapid evaporation of moisture from the surface of the concrete, shrinkage begins and cracking or crazing might result immediately. The unequal swelling of the subgrade might produce local bending of the setting concrete with the production of high tensile stresses at a

time when the concrete has practically no resistance to tension. If the concrete is kept moist during its initial curing period its slight expansion due to moisture will result in a small and harmless compressive stress, although all too frequently contraction begins even in this initial stage due to insufficient moisture. During the curing period there will be daily fluctuations in temperature resulting in expansion during the day with the production of compressive stress and in shrinkage at night with the production of direct longitudinal tension. At the end of ten or fifteen days when the usual curing period has ended, the concrete is allowed to dry and it shrinks with the production of tensile stress due to the frictional resistance of the subgrade. Sufficient longitudinal tension might be produced to form transverse cracks at frequent intervals. Such transverse cracks are inevitable unless they are built into the pavement through frequently spaced expansion or construction joints. In the meantime, during the day, the sun heats the top surface of the concrete thereby expanding it, while the bottom surface remains at a more uniform temperature. The effect necessarily is to warp the pavement slab, cause it to bear on the subgrade at the corners and edges and to rise somewhat along the center. At night the reverse is true, the top surface contracts and the bottom surface remains more stationary. The corners and edges now rise and in many cases actually leave the subgrade. High bending stresses are thus again produced and every day the pavement goes through this cycle of bending. During long-continued periods of wet weather the concrete takes up moisture and expands, and when this expansion is combined with the expansion due to high temperature, the concrete is subjected to high compressive stress longitudinally, so-called "blow-ups" sometimes take place and the pavement heaves in spots away from the subgrade. This phenomenon is especially prevalent in the spring of the year, for this time is favorable to the combined expanding effect of high moisture content and high temperature. The action of freezing on concrete pavements should not be overlooked. Such action is especially noticeable where improper materials or construction methods have been used and it results in the scaling of the surface. The influence of the quality of the cement in the production of such scaling is in need of investigation.

Under traffic the pavement is subjected to very severe bending stresses and tension occurs both at the top and at the bottom of the slab depending upon the position of the load. Concrete is not particularly strong in tension, its value being only from one-eighth to one-twelfth of its compressive resistance. Bending stresses are to be feared because of this inherent quality of concrete to be weak in

tension. It is the practice in most localities to permit traffic on concrete pavements in the very early stages of their hardening. For instance, in some states heavy trucks hauling construction materials are permitted even at the end of 15 days when the concrete has gained little strength in tension, and invariably traffic is allowed on the pavement at the end of 21 days when the bending resistance of the concrete is still small.

It is thus seen that concrete pavements are subjected to a variety of influences which tend to harm them and these influences must be combated by the use of proper materials, proper design and construction methods. In general, the most destructive of these influences are those which produce tensile stresses in the pavement and may be tabulated as follows:

1. Shrinkage of concrete due to drying of moisture.
2. Contraction of concrete due to decrease of temperature.
3. Warping of the slab due to variations in temperature and moisture.
4. Bending due to heavy wheel loads applied frequently.
5. Bending due to non-uniform subgrade support.
6. Excessive compressive stress due to rise in temperature or increase in moisture content or a combination of both.
7. Frost action.

An analysis of the above influences leads us to the conclusion that concrete for highway construction would be even more suitable from a technical standpoint than it is at present:

1. If it developed high tensile strength within a very few days.
2. If it had a very low shrinkage factor under the action of moisture and temperature.
3. If it were even more durable in its long-time resistance to alternations of freezing and thawing, wetting and drying, changes in temperature and repeated loads.

Technically then, concrete having high early tensile strength which would be maintained over a long period of years would be very desirable for concrete pavement construction. Investigations of concrete have shown that strength is very largely dependent on the ratio of volume of water to cement used in the mix and that the so-called "water-cement ratio," assuming a given kind of cement, is the most important factor governing the strength of the concrete. It is logical to infer that the use of a higher strength cement or the use of more cement must likewise lead to higher strength concrete.

Several ways are thus open to the engineers who are seeking concrete of higher strength, such as drier consistency, more cement, better aggregates both in quality and gradation and finally higher-strength cement.

In the light of the previously discussed desirable properties for highway concrete let us inquire into our present standard test requirements for portland cement, so far as that seems possible, to see to what extent they insure the production of cement which will make satisfactory and economical highways.

Fineness.—The specifications require that not more than 22 per cent residue shall be retained on a standard No. 200 sieve. The question arises whether or not it would be desirable to require the use of a finer cement. It has been shown in connection with the work of the Joint Conference on Uniform Methods of Tests and Standard Specifications for Cement, in a report published by Committee C-1 of the American Society for Testing Materials dated July, 1919, that there is only a slight increase in the strength of portland cement with increased fineness, and also a slight increase in the crushing strength of concrete. Thus, 1 per cent reduction in the residue on the No. 200 sieve resulted in about 1.3 per cent increase in the crushing strength of 1 : 2 : 4 concrete at 28 days. No figures seem to be available on the effect on the tensile strength or resistance to bending.

Whether the extra cost of finer grinding is justified by the slightly higher strength obtained in the concrete does not seem to be established. Information is needed on this point. Information is also needed on the effect of fine grinding on the durability of the concrete. The effect of fineness on deterioration in storage is also to be considered.

Soundness Test.—It is a question whether the soundness test as now made is really suitable for bringing to light the qualities undesirable in cement from the standpoint of highway construction. A quantitative test for soundness of cement should be investigated to indicate degrees of unsoundness.

High shrinkage of concrete is undoubtedly due largely to the shrinkage of the cement and it is recognized that different cements shrink and expand differently with moisture changes. Some quantitative test of the volume change of cement might be desirable for judging of its value for concrete road construction.

Tensile Strength.—It would seem that unquestionably the tensile strength of cement must influence the strength of the concrete in tension and cross-bending, and it would seem also that cements having the highest tensile strength should be more suitable for highway concrete than those of low strength. Technically, high early tensile

strength is to be desired, provided the strength will not decrease with age.

It is recognized of course that other factors influence the concrete strength even more than the variation in the strength of the cement. While the available test results are not entirely concordant in showing increased concrete strength with increased strength in portland cement as now tested in the form of 1 : 3 mortar briquettes, the trend is certainly in that direction and this trend is well illustrated by tests with high-alumina and other special cements. In certain localities it would be quite practicable to raise the test limits for tensile strength without inflicting any hardship on the manufacturers and without causing them to change their methods of manufacture or the characteristics of their output. In a very few instances, on the other hand, an increase in the tensile strength requirements would be difficult to meet and, in fact, would bar the product of certain mills. Here again, the technical features of the specifications must be considered in conjunction with the economics of the resulting concrete in the highway.

Highway engineers would welcome cement of higher tensile strength, especially in the early period of hardening provided durability and economy are not sacrificed. Highway engineers would also welcome a strength test that could be made after a few days instead of seven days and they would like a refinement in the tensile test which will eliminate the wide variations in test results which are now possible. A reversion to the neat test has been suggested. Let it be remembered that we are concerned with the mortar and concrete-making properties of cement and that fineness of grinding has a different effect on the neat and mortar strengths.

Time of Set.—The setting time of portland cement under certain conditions might well be regulated so that the concrete would set up faster than at present. At times it is difficult to maintain a smooth finish on a concrete road owing to the slumping of the concrete down grade after it has been finished. A quicker setting cement for this purpose would be desirable and, of course, a quick setting cement would be desirable for cold-weather conditions. Cement complying with the low limits of the specifications, however, should be satisfactory for the above conditions but otherwise the present test limits for time of set are suitable for concrete road construction. Instances are recorded of cements passing the Gillmore test and failing in the Vicat test. We should not be satisfied with our present methods of testing for time of setting. They need more careful control.

It is not felt that any of the other tests for portland cement need discussion so far as concrete pavements are concerned.

OPINIONS OF STATE HIGHWAY TESTING ENGINEERS

In order to gain the ideas of the materials engineers in the state highway departments on the necessity of any changes in the cement specifications, a letter was addressed to them and also to the district engineers of the U. S. Bureau of Public Roads in which the following questions were asked.

1. Specifically, have you information showing whether or not cement which just passes the specifications will or will not make good concrete?
2. Have you any test results coordinating the tests on cement with the strength of concrete made from various brands of cement having various physical properties?
3. Have you any definite ideas as to whether or not the requirements for portland cement should be made more severe?

In brief, the answers to the first two questions revealed very little information. There are some test results showing higher-strength concrete with higher-strength cement, but there seems to be no definite information on the value of concrete made with cement just passing the present standard requirements. Of the thirty engineers replying to the third question, ten were in favor of making the tensile strength limits more severe because they thought that a better quality of concrete would result, while the remaining twenty did not advocate changing the present standard specification. Several of the state engineers were dissatisfied with the reliability of the soundness test, others believed finer grinding desirable. The advisability of a strength test which could be made in a few days was also stressed.

The situation with regard to the portland cement specifications as they apply to concrete road construction might be summed up by the following general statement.

As a whole very satisfactory concrete roads have been built with cement passing our present standard requirements. As a rule, however, most brands of cement fall well within our minimum test limits and some doubt has been expressed whether low testing cement is really satisfactory for concrete pavements because of the high early stresses to which pavements are subjected. It would seem that, technically, high early testing cement should lead to still better construction. Economically this has yet to be demonstrated. If high early strength can be attained with no decided increase in cost and no danger of disintegration, surely it is justified and desirable and is something which cement manufacturers should look forward to. It

is felt that in many cases where cement is testing low, the mill practice is at fault and much better results would be possible if such mills would exercise more meticulous control over their product. There is no question in my mind that the general standard of quality of portland cement can be raised merely through careful manufacture on the part of all of the mills instead of in some of them.

Highway engineers are anxious to secure concrete of high early strength and great durability. In general, they recognize that several ways lead to this end, one of which is the use of cement even more suitable than at present available. Perhaps their goal may be reached by the use of higher quality cements. That is well worth investigating but the making of concrete highways is costing vast sums and technical questions are not alone involved. Caution is necessary that too hasty action be not taken before the full facts have been demonstrated. Several organizations and committees are at work in an effort to improve the quality of cement, the methods of testing cement, and the specifications for its control. These committees are not unmindful of the qualities of cement so much to be desired but they likewise are fully impressed with the difficulties with which they are confronted. They, however, are earnestly trying to ascertain the facts and to use them for improvement of cement and cement specifications.

So to manufacturers I would say there is a growing feeling that a higher quality of cement is desired and that some manufacturers at least can and should improve their product for highway construction.

To highway engineers my message would be that they are justified in their attempt to obtain concrete of higher tensile or cross-bending strength, but the means adopted, whether better cement, more cement, or better concrete control, should be based on established facts and it should be remembered that economic as well as technical questions are involved.

DISCUSSION

Mr. J. G. Bragg[1] (*presented in written form*).—In general we must all agree with the statements set forth in Mr. Goldbeck's paper and it is hoped that some time we will have acquired the ideal cement, proper construction methods, and necessary experience with which we may build the perfect, crackless pavement. If there are any who believe that this will occur in the near future they must certainly lack experience or be optimistic to the point of recklessness. Mr. Goldbeck has given us a very comprehensive résumé of the known causes of deterioration of concrete pavements. In addition to these we are very much concerned with the unknown causes. Of course, it is not fair to attribute all of our pavement defects to the portland cement since there are so many other contributing factors. But certainly it must be admitted that in many cases we have adjoining pavement sections of entirely different character where the only apparent variable is the brand of portland cement used in their construction and this notwithstanding the fact that in both cases the cements complied with all the standard requirements. **Mr. Bragg.**

While Mr. Goldbeck did not directly refer to it he will no doubt agree that uniformity of the pavement structure is of vital importance and it is believed that most highway engineers would have a uniform 3000-lb. concrete throughout the entire length of the pavement in preference to a pavement averaging 4000 lb. per sq. in. with some of the samples showing strengths as low as 2000 lb. In our laboratory we have tried particularly to stress the item of uniformity and with this in mind we have made rather careful observations of the portland cement used by us during the past four years. In view of the fact that so much, and yet so little, is known about the product under discussion it is hardly fair to place the blame entirely upon the manufacturer, but it is remarkable, that there is so great a variation in the quality of materials produced in the same locality and frequently in the same plant.

We have found that most manufacturers with whom we deal produce a fairly uniform product and do it consistently. The minority, and fortunately not many belong to the minority, produce a product which meets the specifications some times but ranges from bad through the various stages of poor and mediocre to good. Our data showed

[1] Senior Testing Engineer, New Jersey State Highway Laboratory, Trenton, N. J.

Mr. Bragg. us that the products of such plants were rejected for all known reasons and it could in no case be said that any were consistently low in strength, unsound, quick setting or otherwise defective. This indicated a lack of proper mill control and not a poor raw material or process. It was noted, however, that the average strengths of these cements were considerably lower than were the more nearly stable products. A compilation of our data convinced us that we could increase our tensile strength requirements by 50 lb. per sq. in. and in no way affect the process of 90 per cent of our producers. It is admitted that the remaining 10 per cent were seriously inconvenienced and it was expected that they would be. We are pleased to be able to say that, measured by our present standards, at least one manufacturer included in the 10 per cent is now furnishing a cement well within the requirements of our new specifications and the uniformity of his product is highly improved.

In order that there be no misunderstanding relative to the number of tests on which we based our conclusions, it should be stated that we are using approximately one million barrels of cement per year. This cement is all bin tested, a sample is taken for each 200 barrels and complete physical tests are made on every sample taken. One chemical analysis is made for each bin on a composite sample made up of all the samples taken. About five thousand samples are tested during the season and the writer has no hesitancy in stating that he sincerely believes that a sample for each 200 barrels is not in excess of the number which should be taken in order to determine the uniformity of the product. We are aware of the apparent lack of uniformity of test methods among the various laboratories and have based our revision on our own tests which we strive very carefully to make according to standard requirements and which we know do not vary appreciably within our own laboratory. In making this revision it is not claimed that such a revision could be made generally throughout the country, nor do we know that it would be advisable to do so even though it were possible. We do claim that we are getting for our own purpose a more nearly uniform product.

We are now conducting a detailed survey of pavement defects and the data thus obtained with the records collected during the past seven years will be available for comparison of materials used, composition, construction methods and design of our highways. The survey will include the area of scaled, spalled or patched surface, also number, kind, and lineal feet of cracks per unit of pavement. Testing and construction records have been kept in such manner that it will be possible to group our pavements under types of subgrade, sections,

aggregates, cement, construction methods, etc. It is hoped that within our own department, at least, the survey will, in a practical way, clear up many points which can now be handled only by theory and conjecture. It may possibly shed some light on this question of the quality of portland cement. Mr. Bragg.

In conclusion it can be said that we sincerely hope the committees who are working on the specifications for and methods of testing portland cement will not lose sight of this matter of uniformity of the product. This is mentioned particularly because of the fact that some members of the committees have said that we are now making too many tests for determination of quality. The writer believes also that highway engineers will not be satisfied with simply a revision of our present methods or with substituting a compression test for the tension test requirements. Quite a lot has been said about the compression test and at the time this discussion is written the writer is not familiar with the results obtained from the cooperative tests for comparison of the tensile and compressive strength which were recently completed by Committee C-1. It was our privilege however to take a small part in this investigation and certainly the results of our tests do not indicate that the cumbersome 2 by 4-in. mortar cylinder, requiring considerably more effort and a larger outlay for equipment can be made with any greater degree of accuracy than the present tensile strength briquette, nor do these tests or others made in our laboratory show that the compression test is more nearly indicative of the concrete-making properties of a portland cement. Mr. Goldbeck implies and it is believed by many of us that additional tests for the determination of properties not now covered by our standard requirements are highly desirable.

We would particularly like to endorse Mr. Goldbeck's advice to highway engineers to let whatever means may be adopted for the improving of portland-cement concrete be based on established facts. It seems this message is also applicable to the cement producers and others interested in portland-cement concrete. Whatever means is adopted must be based on the results of both laboratory and field investigations with due consideration given to economic values.

MR. H. F. CLEMMER[1] (*presented in written form*).—Mr. Goldbeck, in his paper, covers by way of introduction, practically all of the considerations in highway construction that have a bearing on the life and strength of pavement concrete. He has set forth in his opening statements the factors which must be considered in specifying qualities of materials that make up this concrete. Mr. Goldbeck further calls Mr. Clemmer.

[1] Engineer of Materials, Illinois Highway Division, Springfield, Ill.

Mr. Clemmer. attention to the difference between pavement concrete and that used for other purposes and emphasizes the necessity for the development of strength resistant to cross bending and resultant flexural stresses.

Whether or not, considering our present information on cement and concrete, we can successfully remodel our specifications to provide better cement is a well-suggested question. Remembering that we have a commercial as well as technical problem to consider in the event of a change in requirements, definite action should certainly be preceded by careful consideration based on proved results of tests. Mr. Goldbeck has very wisely been conservative in his discussion and has indicated the caution we should use in altering specifications.

It cannot be doubted that great improvements are possible in the quality of portland cement. At the same time it is well known that we have not yet become overly proficient in controlling the quality of concrete. Many variables, having as important a bearing on the strength and life of concrete as the variation in the quality of cement used, are known to us and include all those which Mr. Goldbeck has tabulated. When we know that all these variables can be successfully regulated and our concrete lacks no other refinement or improvement than the use of better cement then we can well worry about what the mills are shipping to our jobs. This does not infer, however, that we should not adhere rigidly to our present specifications.

The Illinois Highway Laboratory has conducted tests on twelve different brands of cement used in Illinois for highway construction. These cements were identified, for purposes of comparison, by letters A, B, C, etc.

These researches indicate that there seems to be wanting in our present specifications, a test, which, by either an accelerated physical or chemical method would reveal the rate of hardening. In comparing the results of our tests on the relative strengths of these twelve brands of cement, at ages up to and including one year, there is revealed the significant fact that the rate of hardening (or increase of strength) of the various brands develops curves of different trends. Especially noteworthy are the strengths developed by two of these brands. At the end of the respective test periods (7, 28, 60, 90, 180, 360 days), Brand E exhibits compressive strengths greater by 63, 62, 40, 24, 0.5, and −1.0 per cent than those shown by Brand C. In the physical tests there is nothing that would indicate these final results. The initial and final sets vary by 30 minutes, the fineness by only 1.5 per cent; and the normal consistencies are practically identical (22.6 and 22.5 per cent). The chemical analysis reveals some differences, Brand E showing higher alumina and a lower silica content.

Although both these brands meet our present specifications, it is the opinion of the writer that at the end of ten years a considerable difference would be apparent in the durabilities of the concretes into which these cements had been incorporated. Mr. Clemmer.

There exists a need for an accelerated chemical method for determining the quality of cements, as well as the study of the effect of the basicity of cement and its relation to chemical composition. The mere radical form of the chemical composition of a cement is not sufficient, we should have a knowledge of the actual constituents. Such a test would be a substitute for our present soundness test which, as stated by Mr. Goldbeck, could well be replaced by some quantitative determination that would indicate similar qualities.

Finally, it is the opinion of the writer that every testing engineer accepts the responsibility of securing, for pavement concrete, cement that not only passes our present requirements but is of the highest quality possible under those requirements.

MR. C. L. MCKESSON[1] (*by letter*).—Some time ago we submitted some data for the use of the committee which has in hand the matter of changes in standard requirements for cement. We are very earnestly advocating a compressive strength requirement of 2000 lb. per sq. in. at 7 days and 3000 lb. at 28 days. All of our brands of cement produced in this section readily comply with the present tensile strength requirements. As a matter of fact, we have a regular practice of making 3-day tensile strength tests and our cements always meet the 7-day requirements in three days. We have found, however, a very wide variation in the compressive strength of concretes made with the nine brands of California cement. We have a feeling that cooperation on the part of the manufacturers will enable us to secure uniformly high compressive strengths. Mr. McKesson.

MR. H. S. MATTIMORE.[2]—Mr. Goldbeck in his paper has covered this subject in a very comprehensive manner so far as indicating the service requirements of concrete in concrete pavements. Since these service requirements are about as severe as any to which concrete in any structure is subjected, he definitely draws the conclusion that the best quality of cement obtainable should be used in this class of work. Mr. Mattimore.

I believe all those who study concrete pavements will agree with him in this conclusion, and also agree with the statement that in many cases the mill practice is at fault and that much better results would be possible if better technical control were exerted. We find in our

[1] Materials and Research Engineer, California State Highway Commission, Sacramento, Calif.
[2] Engineer of Materials, Pennsylvania State Highway Department, Harrisburg, Pa.

Mr. Mattimore.

work that the most reliable cements are being obtained from companies or mills where the chemical branch has the final authority regarding methods of production. The making of cement is an intricate process, and our past experience would indicate that in order to assure good results quality control is the vital feature and should always come before increase of production.

Soundness Test.—The testing for soundness depends so much on the personal equation in the final report on the degree of soundness, that some definite quantitative test for soundness would be a step forward. It is my understanding that many of the European standards, especially the British standard specifications for cement, have a maximum allowable expansion during the steam test.

Time of Set.—I would hesitate to recommend the shortening of the setting time for cements, because under field conditions a quick-setting cement when used in concrete may cause rapid dehydration with its accompanying defects, such as excess shrinkage due to the concrete being subjected at various stages to very severe conditions of high temperature and low humidity. In so far as the comparison of the Gilmore and Vicat tests is concerned, the Vicat is the more positive and can be better controlled. What we are seeking in the highway field is not a quick setting cement so much as an early hardening one.

Tensile Strength.—It would be desirable to obtain high early tensile strength provided, as mentioned by Mr. Goldbeck, that the strength will not decrease with age. I would add a further provision, that any measure taken to secure a high early tensile strength should not be at the sacrifice of the durability of the concrete. If this high early tensile strength can be secured by finer grinding of the raw products, a harder burning and finer grinding of the finished product, it would probably be a step forward, but a danger might exist in overliming the mix or changing the proportions of raw materials so as eventually to lead to a non-durable product which might not be detected by our present methods of test. Another factor that we have to consider in securing cements of high early tensile strength is that such cements are subject to a greater volumetric change during hydration than the normal portland cement.

Some years ago we were striving to bring the coarse and fine aggregate used in concrete up to a point where we felt that they were equal in quality to the cement used, or at least were not the main cause of poor concrete. I believe that the large construction projects, especially in the highway field to-day, have arrived at this point where they have excellent control of the coarse and fine aggregate and

the manipulation of the concrete itself. So it seems that in order to improve the quality of concrete highways the quality of the ordinary cement will have to be improved. Mr. Mattimore.

MR. THADDEUS MERRIMAN.[1]—The author of this paper is to be congratulated in having so concisely and so forcibly indicated the real need in concrete, namely, greater tensile strength. The compressive strength of concrete is far from being an index of either quality or durability. The speaker doubts whether any structure of concrete has ever failed in compression. Local failures in compression are often seen, for instance, as between two sections of a retaining wall where no provision for expansion has been made, but a real honest-to-goodness failure in compression is unknown except in the testing machine. On every hand, however, are to be seen failures in tension. We have been concentrating our studies on the compressive strength, possibly because the results tend to lull us into a sense of security as this strength keeps on increasing with time. The older a concrete specimen is, the greater is its compressive strength. Its tensile strength, however, is probably no greater at the end of a year than it was at the end of 7 days. The speaker recently made a few experiments designed to show the increase in tensile strength in concrete. In these experiments a number of concrete fence posts were tested as cantilever beams. The results are as follows: Mr. Merriman.

Post	Load in Pounds at Appearance of First Crack		
	24 Hours	7 Days	28 Days
No. 1	246[a]	207	220
No. 1*A*	112	156	163
No. 7	214	163	227
No. 8	342	201	246
No. 16	92[b]	143	233

Posts 1, 7, 8 and 16 were cured wet under burlap for 24 hours(±), then in damp sand.

Posts 1*A* were cured in the open air.

Posts 7 were mixed with 1.25 per cent of calcium chloride.

Posts 8 were mixed with 2.5 per cent of calcium chloride.

a This post was 28 hours old at time of test.

b This post was 20 hours old at time of test.

In another series of 10 posts, the average load at which the first crack appeared was 202 lb. at 7 days and 231 lb. at 28 days. By thoroughly cleaning the gravel aggregate, which was already very clean, the preceding figures were increased more than 20 per cent.

All of the posts were of the usual D shape, tapering from $3\frac{1}{2}$ by $3\frac{1}{2}$ in. at the top to $5\frac{1}{2}$ by $5\frac{1}{2}$ in. at the bottom and 7 ft. 4 in. long. Each

[1] Chief Engineer, Board of Water Supply, City of New York.

Mr. Merriman. post carried four $\frac{1}{4}$-in. deformed bars. They were tested as cantilevers, curved side down, the fulcrum being 4 ft. 3 in. from the small end, the load being applied 4 in. from the small end. The concrete was mixed 1:1.3:2.7. The fineness modulus of the sand was from 2.82 to 3.08, that of the gravel from 6.49 to 6.83 and that of sand and gravel combined averaged about 5.37. Four and three-quarter gallons of water were used per bag of the cement, which was Nazareth. The results indicate the importance of the curing condition as between posts 1 and 1*A* and show that the apparent high early strength, due to the use of calcium chloride, is not a permanent asset. Many other posts were also tested under various curing conditions but the results are not pertinent to the present discussion.

The tensile resistance, as indicated by the appearance of the first crack, was practically as great at 24 hours as at either 7 or 28 days. Mr. Goldbeck states that heavy trucks go onto many concrete pavements at the end of 15 days and that the pavements apparently successfully withstand the bending stresses so induced. As far as bending alone is concerned, it might be inferred from the above table of tests that the road would stand up under such loads at a much earlier time. Of course if such a load went onto a concrete highway within 24 hours, there would be local crushing due to failure in compression directly under the load, but so far as the bending of the slab as a whole is concerned, the tensile resistance developed would be a very large part of the ultimate tensile strength of the slab. In any mass of concrete there is no relation between the tensile strength of the cement and the tensile strength developed by the concrete. The tensile strength of concrete depends very largely on the adhesion which the cement develops along the planes and surfaces of the aggregate. If the cement does not adhere to the aggregate, the tensile strength of the concrete will be very low. There are many factors and causes which control the development of the adhesion between the cement and the aggregate. We all know that on the bottom side of each piece of aggregate the adhesion is very small; that it is better on the top and sides of the aggregate and that everywhere it is a long way from being perfect. By cleaning the surfaces of the aggregate so as to make them receptive to the cement, the tensile strength can be very largely increased. The compressive test on concrete means, to my mind, no more than does the compressive strength of two bricks placed one on top of the other. A system of this kind will develop a very substantial compressive strength, but its tensile resistance, depending on the direction of the pull, is no more than the weight of one brick.

In closing I desire again to compliment the author on having so frankly stated the requirements which portland cement for use in concrete should meet and if there should be included in the investigations suggested by him a study of different cements with respect to their ability to adhere to the surfaces of the aggregates, the sum total of our knowledge would be very greatly enhanced. Mr. Merriman.

MR. DUFF A. ABRAMS.[1] It should be pointed out that the characteristics of concrete made from portland cement or from any cement depends on a great many factors besides the properties of the cement. With a poor or mediocre cement, we may make a concrete of very excellent quality, or, on the other hand, with an excellent cement we may make an altogether rotten concrete. We know that the ratio of water to cement is a controlling factor in the final result, regardless of the quality of cement used. In our experimental studies of concrete, we have found the compressive strength to vary from 500 to 3500 lb. per sq. in. at 28 days for the same cement and aggregate, due solely to variations in the quantity of mixing water. The concrete which gave 500 lb. per sq. in. was not nearly as sloppy as much of the concrete which goes into important work. Mr. Abrams.

With a cement meeting standard specifications, we can make concrete with a very great variety of qualities, depending on other factors such as the time of mixing, the method of placing, curing conditions and so on. In making concrete we are manufacturing a structural material of which cement is one constituent, but proportions of materials, manipulation, and subsequent treatment are also most important factors.

I wish to emphasize the fact that the compressive strength of concrete is a good measure of its resistance to other stresses and to destructive agencies such as weather, sulfate waters, etc.

MR. N. C. JOHNSON.[2]—I have listened with very great interest to this discussion, but it seems to me that when we are talking about concrete we ought to have a mental representation in these meetings from an utterly neglected class, or two or three classes, with which are vitally concerned the character and behavior of concrete. We ought to have here a large delegation of laborers, because they are the ones who tell us what we are going to get; we ought to have a large delegation of foremen, because they determine what the laborers give us; we ought to have a few hard-boiled superintendents; and we ought to have a few general managers who notify the superintendents that "If you don't get that done by September 1 you are going to hear Mr. Johnson.

[1] Professor, In Charge of Structural Materials Research Laboratory, Lewis Institute, Chicago.
[2] Member of Firm, Hool and Johnson, New York City.

Mr. Johnson.

from me." Now, do not forget these things when we are talking about cement.

Here in these meetings we discuss tests of small briquettes of cement and mortar. When applying these tests to the world-wide field of concrete, to millions of yards, to gigantic constructions, we must consider a lot of factors usually neglected. When we are dealing with a laboratory specimen, we are dealing with only a very small batch, and we everlastingly puddle it to get it anywhere near uniform. But we rarely do that in commercial work. And when we are dealing with standard briquettes, the mixing is out of all proportion to the practices of construction work. Yet these test pieces are assumed to guarantee our large structures. Further, in such laboratory work, we have none of the factors that are entering into commercial work. For instance, in most forms there is a mass of steel—frankly, I have seen such quantities of steel in a form that it was almost impossible to pour the concrete. Into such a form concrete is being poured down 20 ft. or so from buggies, or from chutes; and in the concrete is perhaps every size of stone from $\frac{1}{4}$ up to $1\frac{1}{4}$ or $1\frac{1}{2}$ in. More or less of these falling stones hit some brace, some form tie, or some steel; and when the $1\frac{1}{2}$-in. stones hit, they will bounce over here, the $\frac{3}{4}$-in. stones bounce a little less and the $\frac{1}{4}$-in. stones do not bounce at all, they just dribble down. That separation happens in every form, but we do nothing commonly in our practice to readjust such mixes. But there is a time-honored rule which is observed quite religiously, and that is to spade it back from the forms so as to bring to the surface as much as possible of the cement that ought to be in the mass, so we unbalance the mix as far as we possibly can and put different material at the surface to crack up.

Then, consider for a moment the feature of design. Design on paper is acceptably standardized, that is, our standards are such that no matter how much that design is maltreated in the field, in the placing of the steel and the concrete, we do not have collapses. I have not seen an actual collapse of concrete due to design, but the maltreatment of design is—well, I don't know whether it is pitiable or laughable. Our engineers figure out their reinforcement assuming the steel to be a definite distance from the neutral axis and they put in the amount of steel that is called for by the standard design, but in the field, the steel is just as apt as not to be on the neutral axis, or perhaps above it, and yet the structure doesn't fail. I think this ability to stand abuse is a remarkable tribute to concrete.

Mr. Goldbeck spoke of the possibility of increased fineness in cement and its presumed value. If you will permit me, I should like

to reverse that and put the question this way: "How fine can portland cement be ground without detriment to its qualities?" Now if you will please consider that seriously I think you will find a wide field of productive thought. Mr. Johnson.

Mr. Merriman's statement that he has found that the tensile value of portland-cement concrete is developed within 24 hours, is significant. I am not in a position either to disagree or to agree, although it seems almost incredible to me from the nature of colloids and the nature of portland cement, but if he is right it is a most significant thing that should be thoroughly investigated.

In that connection, let us consider a pavement surface. We will assume that the pavement is made from a perfect cement, and a perfect mix, perfectly placed, which, of course, is an unwarranted assumption in view of the commercial art of to-day. We have, however, in all pavements, a very high evaporation factor which means that a great deal of cementitious material in solution is coming to the upper surface, so that by evaporation large quantities of portland cement in solution are being brought to the surface and concentrated there. These concentrations of cement are different from the body of the mass and possess totally different properties in their expansion, absorption, etc. Furthermore, we have another thing happening, namely, the reaction between the hydrates of alumina and lime and the carbon dioxide of the air. Hence, a large part of the surface is not a normal concrete surface but is a very measurable layer of an entirely different material—calcium carbonate and other products with alumina and iron. I am not able to say just exactly what these materials are, but the result is this: we get a very pronounced shrinkage at the exposed plane which is entirely distinct from evaporation and which is very apt, be it visible or invisible, to result in the three-branch cracks which distinguish all tensional surfaces in concrete. Now, so strong is that skin between the three branches, in that area, that the rupture frequently is as deep as a foot within the first few hours of placing the concrete, and such a crack never heals. Such surfaces are subject to frost, because these lesions are of microscopic size and draw water and other things in.

These effects can be artificially produced in any laboratory, and quite independent of evaporation. If, for instance, you will take a shallow box, say with 3-in. sides above the concrete, cast a concrete slab and flood the box with carbon dioxide, you will find that cracking effect within a half hour. I have taken motion picture photographs of it happening, and I showed the photographs some seven years ago. The cracking is just like a flash of lightning. When it starts in one

Mr. Johnson.

place, away it goes over the whole surface, and then it grows more slowly, and continues to grow.

Another thing we should consider in discussing the quality of cement is that at the formative period portland cement can be made good or bad just as readily as though it were well or poorly compounded and well or poorly burned at the mill. Portland cement is anhydrous; when water is added, the hydration products are formed. Now if you add something during that formative period, you can make or mar the concrete. Thus, if you add too much lime either at the mill or in the hydrating period, the composition of the cement falls outside the setting region entirely. Or, if you subtract lime, the same thing can occur. If you add alumina in some proportion such that the composition is outside the setting region, the cement is either rendered very sound or very unsound, just as you will. Many of the materials that will alter the cement equation during the formative period are actively contained in every aggregate except silica. Thus, considering the addition of alumina, suppose a shale aggregate is used. It is more or less soluble. That solubility affects the cement during the formative period and you may have a failure. Another thing we are commonly neglecting is that the materials of commerce are contaminated with grease. Grease may be of various kinds and various natures, and it has an effect according to its composition. Mr. Merriman has touched upon that, and there our experience runs parallel, in that a cleansing of the aggregate makes a considerable difference in strength values.

I would say in conclusion, let us thoroughly investigate cement, but by all means let us not lay all the difficulties at the door of the product as it is to-day.

Mr. Bragg.

MR. BRAGG.—There is one other thing I want to bring out. It is a thing I tried to stress in my discussion, and that is the matter of uniformity. Take, as an example, reinforcing steel—we have structural, intermediate and hard grades. Now the early strengths of portland cement are important. Why should we simply require portland cement to have a tensile strength of 200 lb. per sq. in. in 7 days? It may be 200 or it may be 325 lb. That's a thing that we were trying to get at when we changed our requirements on portland cement. I see no reason why we should spend all this time on aggregates, on construction methods, on inspection, etc., and overlook an item of that kind.

Mr. Fitch.

MR. T. A. FITCH.[1]—Mr. Goldbeck's paper brought out openly and frankly the known unreliability, particularly of the soundness test, and the unscientific variation we have all experienced in the

[1] Testing Engineer, City Engineering Department, Los Angeles, Calif.

tension test. We can corroborate the fact that these two features alone are enough to make it necessary to change our methods of cement testing, and I wish to state that in our work, years ago (5, 6 or 7 years, to be specific) we completely abandoned any attempt to rate cement by tension tests, and even in that early day attempted (and I may say successfully attempted) to standardize the compression test, so that we have a means of at least more accurately gaging the value of the strength of cement. At the same time, we deliberately dropped the soundness test as a requirement for acceptance, and, even more startling still, dropped the fineness test. In the place of this we carry setting time and strength, as I have said, and, paramount, the requirement that the strength *in concrete* shall be acceptable. This **Mr. Fitch.**

TABLE I.—COMPARISON OF FLEXURAL AND COMPRESSIVE STRENGTH OF CONCRETE.

Flexural tests of 7 by 10 by 38-in. plain concrete beams loaded at third points of 36-in. span.
Compression tests of 6 by 12-in. concrete cylinders.
Mix 1:4 by volume.
Aggregate: sand and pebbles.
Relative consistency 1.10 (equivalent to slump of about 3 to 4 in.).
Specimens cured in moist room and tested damp.
Each value is the average of 5 tests made on different days.

Size of Aggregate	Modulus of Rupture, lb. per sq. in.				Compressive Strength, lb. per sq. in.			
	7 Days	28 Days	3 Mos.	1 Year	7 Days	28 Days	3 Mos.	1 Year
0–No. 4	125 (50)[a]	250 (100)[a]	370 (148)[a]	425 (161)[a]	430 (42)[a]	1010 (100)[a]	1620 (160)[a]	2100 (206)[a]
0–$\frac{3}{4}$ in.	290 (64)[a]	455 (100)[a]	595 (131)[a]	640 (141)[a]	1040 (49)[a]	2110 (100)[a]	2930 (139)[a]	4490 (212)[a]
0–1$\frac{1}{2}$ in.	420 (76)[a]	550 (100)[a]	810 (147)[a]	880 (160)[a]	1410 (55)[a]	2580 (100)[a]	3590 (139)[a]	5000 (194)[a]

[a] Percentage of strength at 28 days.

is in line with Mr. Godbeck's final recommendation, wherein he gives three ways in which to progress, the third and last being that we find means of better concrete control. This seems to me to be the solution, and the concrete highways of Southern California are the evidence that it can be done under these conditions.

MR. A. T. GOLDBECK.—I have very little else to say, for the subject has been pretty well covered. Mr. Merriman made a rather startling statement, startling to me, at any rate, namely, that the cross-bending strength of concrete is the same at 24 hours as it is at later periods. I should like to ask Mr. Abrams whether he has made any extended series of tests along that line and what his experience has been. **Mr. Goldbeck.**

MR. ABRAMS.—I must confess that Mr. Merriman's statement that the tensile strength of concrete as measured in beams does not increase with age, was very startling to me. **Mr. Abrams.**

Mr. Abrams. We have made a great number of tests of plain concrete in cross-bending, as well as a few tests in direct tension. The results of an early investigation are reported in our Bulletin 11 on "Flexural Strength of Plain Concrete" (1922). Four later series of tests give additional information on the modulus of rupture of plain concrete of a wide range of mixtures, consistencies, type and size of aggregate, curing condition, age at test, etc.

The parallel flexural and compression tests of concrete at ages of 7 days to 1 year given in the accompanying Table I are from Table IV, Bulletin 11.

So long as the concrete remained damp, there was a continuous increase in both modulus of rupture and compressive strength; in no case was there a retrogression in strength with age. In the above tests, the modulus of rupture at 7 days was 50 to 76 per cent of the 28-day values and at 1 year 141 to 161 per cent.

The compressive strength of concrete increases more rapidly than the modulus of rupture. In other words, there is not a constant ratio between modulus of rupture and compressive strength; the ratio appears to be a function of the *strength* of the concrete, regardless of the reason for the particular strength. Following are average ratios calculated from five series of tests made over a period of 5 years:

Compressive Strength, lb. per sq. in.	1000	2000	3000	4000
Modulus of Rupture, percentage of Compression.....	24	18	16	14

The following are typical values of modulus of rupture of plain concrete made outdoors at Sacramento, Calif., in a recent study of curing of concrete: 3 days, 340 lb. per sq. in.; 7 days, 435; 14 days, 490; 28 days, 505; 3 months, 565 lb. per sq. in. These beams were of the same size as those mentioned above; after removal of forms at 16 to 24 hr. they were kept covered with wet earth for 3 days and then exposed to the hot, dry summer air.

I have seen frequent evidences of confusion in discussions of properties of concrete due to basing conclusions on briquette tests. It is typical of briquette tests that the strengths show retrogression after 28 days to 6 months. The reason for this lies in the shape of specimen and does not reflect a property of cement or concrete. A uniform distribution of load across the section is not obtained, consequently, as the cement or mortar gets harder, the application of the load to the briquette sets up a tearing action which has the effect of producing failure at a lower load, although the mortar is really much stronger than at earlier periods.[1]

[1] See "Effect of Age on the Strength of Concrete," by Duff A. Abrams, *Proceedings*, Am. Soc. Testing Mats., Vol. XVIII, Part II, p. 317 (1918).

MR. MERRIMAN.—In using the words tensile strength when referring to concrete I do not refer to the tensile strength developed by the cement itself, but rather to the strength of adhesion between the cement and the surfaces of the aggregates. The beams which we tested were fence posts. All similar previous tests on beams have been made when the specimens were anywhere from 7 to 100 days old. There is no record of any made as early as 24 hours. When the suggestion for tests at 24 hours was first advanced, no one believed that any result whatever would be obtained. The actual results, however, tell their own story. If adhesion between the cement and the aggregate is not secured at an early time, it can never thereafter develop. The adhesion between a glue and a surface is just as great while the glue is soft as it is at any time thereafter. A glued joint, however, cannot develop full strength until the glue has set and developed its own structure sufficiently to transmit the stress to the surface of contact. Just so is it in concrete. The cement acting as a bonding glue makes contact with the surfaces of the aggregates and that contact, when once made, can never become better or stronger. As the cement itself develops rigidity, the compressive strength of the concrete will increase but its resistance to tension will, unfortunately, remain essentially constant. **Mr. Merriman.**

It is exceedingly difficult to conceive of a function so simple as that commonly implied by the words "tensile strength of concrete." The resistance of a concrete to a tensile force, as measured in pounds per square inch, is a composite value which embraces, among others, the following elements (1) the cohesion between the cement particles (2) the adhesion between the cement particles and the surfaces of the aggregate (3) the shearing strength of the cement particles along the surfaces of the aggregate and (4) the reinforcing value of the aggregate acting across planes at right angles to the direction of the tensile force. The true actual resistance of a non-reinforced concrete in either tension, shear or compression seems to be the load which produces the first crack and this is the criterion on which our recent investigations have been based.

MR. R. B. YOUNG[1] (*by letter*).—It may be of interest to note that Committee C-1 on Cement, of which the writer is secretary, has taken cognizance of Mr. Goldbeck's paper and the discussions thereof and has referred them to its several sub-committees for study and action. **Mr. Young.**

[1] Senior Assistant Laboratory Engineer, Hydro-Electric Power Commission of Canada, Toronto, Ontario, Canada.

A STUDY OF TEMPERATURES IN HIGH-ALUMINA CEMENT AND METHODS OF CURING

By H. S. Mattimore[1]

Synopsis

During some experimental work undertaken by the Pennsylvania Department of Highways, to test the efficiency of a high-alumina cement for the purpose of making early high-strength concrete, it was noted that defective surface conditions resulted when such concrete was cured by methods which had been effective in the curing of portland-cement concrete. Accordingly, several series of test slabs were cured under two conditions, one, a combination of wet burlap and ponding curing, and the other ponding curing alone. Provision was made in this test for the measurement of the temperatures in the slabs at various periods during curing. The surface conditions are distinctly better in the case of the latter method of curing.

The effect upon this phenomenon of the age of the high-alumina cement has been studied.

Comparative tests of mortar and concrete of high-alumina and portland cement comprising tension and compression tests of mortar and transverse compression tests of concrete are given.

During the construction by the maintenance forces of the Pennsylvania Department of Highways, of a short section of an experimental road surfaced with high-alumina-cement concrete, a series of test specimens was cast from concrete being placed from a four-bag capacity concrete mixer. These specimens were in the form of beams 6 in. wide, 8 in. deep and 40 in. in length, for modulus of rupture tests, and 6 by 12-in. cylinders for compression tests. In order to secure comparative data, similar specimens of portland cement concrete were cast from several batches made in the same mixer.

The results of a series of laboratory tension and compression tests on high-alumina cement and portland cement, using 1:3 mortar specimens, are given in Table I. Table II gives the results of the tranverse and compression tests on the concrete field specimens of both cements.

A comparison of the concrete tests made with these cements demonstrates that high-alumina cement produces concrete having considerably higher tranverse and compressive strength at 24 hours than portland-cement concrete at 28 days.

[1] Engineer of Materials, Pennsylvania State Highway Department, Harrisburg, Pa.

It was observed in the examination of the test beams, prior to and after testing, that the surface developed a soft or scaled top, varying from the so-called dusting surface to a distinct scale $\frac{1}{8}$ in. or greater in depth. This condition was characteristic of all specimens but was more readily observable in the beams than in the cylinders,

TABLE I.—COMPARATIVE TESTS OF HIGH-ALUMINA AND PORTLAND CEMENTS.

	FINENESS, PER CENT RETAINED ON NO. 200 SIEVE	INITIAL SET (VICAT), MINUTES	HARD SET (VICAT), MINUTES	SOUNDNESS
High-Alumina Cement..	3.7	675	735	O. K.
Portland Cement.....	18.4	242	347	O. K.

1 : 3 OTTAWA SAND MORTAR

AGE	TENSILE STRENGTH, LB. PER SQ. IN. HIGH-ALUMINA	TENSILE STRENGTH, LB. PER SQ. IN. PORTLAND	COMPRESSIVE STRENGTH, 2 BY 4-IN. CYLINDER, LB. PER SQ. IN. HIGH-ALUMINA	COMPRESSIVE STRENGTH, 2 BY 4-IN. CYLINDER, LB. PER SQ. IN. PORTLAND
24 hours.....	315	75	4 218	252
	345	75	5 429	285
	330	75	....	...
Average.....	330	75	4 823	269
48 hours.....	455	155	6 430	680
	440	140	5 750	704
	...	130	....	...
Average.....	448	142	6 090	692
7 days.....	540	230	6 448	1 445
	560	265	6 526	1 464
	535	235	....	....
	525	250	....	....
Average.....	540	245	6 487	1 455
28 days.....	515	365	6 951	2 495
	480	350	6 761	2 457
	450	305	....	....
Average.....	482	340	6 856	2 476

due to their larger surface area; in fact there was much doubt that it would be detected in a 6 by 12-in. cylinder without critical examination. This condition apparently had little or no effect on either the transverse or compressive strengths at the periods recorded in Table II, but such a surface condition is detrimental in floors and concrete road slabs and may affect the durability of any structure.

TABLE II.—COMPARATIVE TESTS OF HIGH-ALUMINA CEMENT AND PORTLAND-CEMENT CONCRETE.

FINE AGGREGATE		COARSE AGGREGATE	
Passing No. 100 Sieve..	1.5 per cent	Passing ¼-in. Screen...	10.0 per cent
Passing No. 50 Sieve..	17.2 "	Passing ½-in. Screen...	15.6 "
Passing No. 20 Sieve..	62.1 "	Passing ⅜-in. Screen...	20.6 "
Passing No. 4 Sieve..	90.0 "	Passing ¾-in. Screen...	23.7 "
Passing ⅜-in. Screen....	100.0 "	Passing 1 -in. Screen...	33.1 "
Loss by Washing.....	1.5 "	Passing 1¼-in. Screen...	48.7 "
Colorimetric Fig. No....	1	Passing 1½-in. Screen...	70.6 "
Compressive Strength		Passing 2 -in. Screen...	96.9 "
Ratio, 1:3 by Volume:		Passing 2½-in. Screen...	100.0 "
7 Days.....	132.7	Wear.....	3.0 "
28 Days.....	121.9 "	Toughness.....	14

1:2:3 CONCRETE

AGE	CEMENT	MODULUS OF RUPTURE,[1] BEAMS, WIDTH, 6 IN., DEPTH, 8 IN., SPAN, 24 IN. (CENTER LOADING) Ultimate Load, lb.	Modulus of Rupture, lb. per sq. in.	COMPRESSIVE STRENGTH,[1] CYLINDERS 12 BY 6 IN. Compressive Strength, lb. per sq. in.
24 hours	High-Alumina	10 200	956	5 161
		9 940	932	4 736
		....	...	5 623
	Average	.....	944	5 173
48 hours	High-Alumina	9 980	936	4 947
		9 840	923	6 422
		....	...	5 219
	Average	.....	929	5 529
7 Days	High-Alumina	10 630	997	5 845
		9 050	848	5 550
		....	...	6 245
	Average	.....	922	5 880
21 Days	High-Alumina	13 050	1 223	6 401
		10 680	1 001	5 929
		....	...	7 109
	Average	....	1 112	6 480
21 days	Portland	6 280	589	2 758
		5 370	503	3 471
		....	...	2 471
	Average	.....	546	2 907

[1] Specimens fabricated in field from concrete deposited in place from mixer.

Soft and scaly surface conditions on mortar and concrete slabs may be caused by the materials used in the fabrication or by the method of manipulation, but they are often traceable to curing conditions. Since well-graded coarse sand and clean stone of good quality were used in these specimens, material was eliminated as one cause and the curing was given the main consideration. The test specimens under discussion were cured under wet burlap applied 1 to 3 hours after

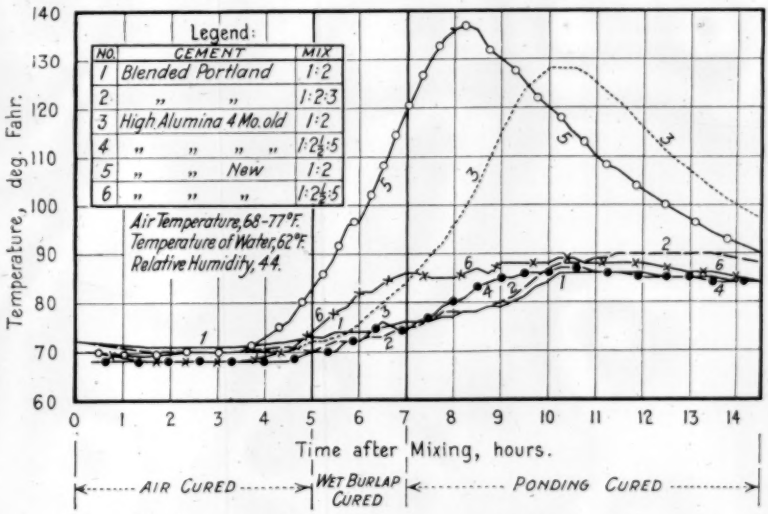

FIG. 1.—Showing Hydration Point of Portland and High-Alumina Cement, Burlap and Ponding Curing. Mixes as Shown.

the concrete was placed, a method which has been found efficient and which is used extensively for concrete road slabs and floors.

Several series of test slabs 24 in. long, 9 in. wide and 3½ in. deep were made, and a short test tube filled with mercury was placed in the center of each slab extending within ½ in. of the bottom. Frequent temperature readings were taken on each slab by placing a thermometer in the mercury.

Fig. 1 shows the periodic temperature readings on high-alumina and portland-cement mortars and concrete with combinations of wet burlap and ponding curing. It was thought that storage might affect the high-alumina cement, so both fresh and stored cements were used. All specimens of high-alumina cement mortar and concrete in these series show a distinct defective surface condition.

Fig. 2 shows a similar set of specimens with air and ponding curing. It is readily observed in comparing these two figures that burlap curing causes a much higher ultimate temperature than where ponding curing alone is used. This is especially noticeable in the mortar specimens Nos. 3 and 5 of both figures.

All specimens illustrated in the series of Fig. 2 had a more normal surface condition than those illustrated in Fig. 1. Specimens Nos. 5 and 6, made with new cement, had a harder surface than Nos. 3 and 4 which were made with cement stored 4 months in bags. Pond-

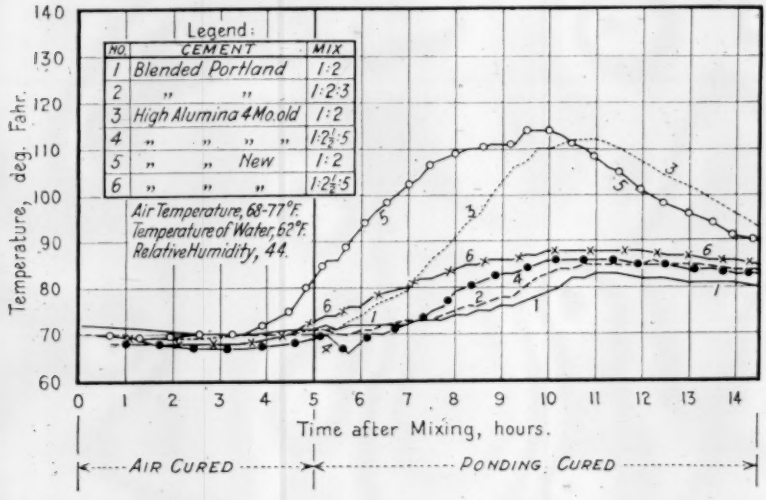

FIG. 2.—Showing Hydration Point of Portland and High-Alumina Cement, Ponding Curing. Mixes as Shown.

ing at the 5-hour period was effective on the new cement, but was too early a period for the cement which had been air cured in bags. It can be observed on both Figs. 1 and 2 that the rapid rise in temperature of the fresh cement starts at 5 hours while the stored cement is retarded until the 7-hour period.

A later series of tests,[1] which are not included in this paper, distinctly shows that ponding at the 7-hour period, under the temperature and humidity conditions of the laboratory, which were the same as given in Figs. 1 and 2, is effective and results in a good hard surface condition for both new and stored cement. This was ascertained for the purpose of arriving at a period when water curing may

[1] This series is a duplicate of those shown in Figs. 1 and 2 with ponding used at the 7-hour period.

be started without having to take into consideration the age of the cement. The time period for applying water will vary with temperature and humidity, but it is indicated on the specimens by a surface stiffening, when little or no indentation is made on the surface by a slight pressure of the fingers, or by a drying out appearance.

In the construction of portland-cement concrete highways under certain atmospheric conditions, it is necessary to apply wet burlap at early periods, from 1 to 3 hours after placing, to avoid surface checks or cracks which occur during air curing due to rapid dehydration. It will require further investigation under field conditions to study the action of high-alumina-cement concrete during early periods of air curing. The data and observations made in the study upon which this paper is based indicate that such air curing will be necessary for from 5 to 7 hours.

The following definite conclusions are drawn:

1. High-alumina-cement has compressive and transverse strengths at 24 hours greater than those of portland-cement concrete at 28 days.

2. Wet burlap curing, effective for portland-cement concrete, increases the temperature during hydration and produces a defective surface when used with high-alumina-cement concrete.

3. Application of moisture at too early stages, either by sprinkling, ponding or covering with wet burlap, produces a dusty or scaled surface.

4. Water curing should be started when hydration is well under way as indicated by a rise in temperature. This can be detected by a stiffening of the surface and a drying out appearance. Under laboratory humidity and temperature conditions, the safe period was found to be 7 hours after mixing.

5. Storage or air curing in bags reduces the high temperature during hydration and also delays the hydration. After four months storage in bags the hydration was delayed about two hours.

DISCUSSION

Mr. Conwell.

MR. E. L. CONWELL[1] (*by letter*).—Mr. Mattimore's paper is of outstanding interest in that it throws light on the sharply different behavior of alumina cement from portland cement during the curing period. Several hundred tests of mortars and concretes of alumina cement have been made in the writer's laboratory during the past three years. The earlier tests were made on lots of European alumina cement that were imported for experimental purposes. Most of the tests of the past eighteen months have been on American-made alumina cement. With the exception of a sample of electric furnace alumina cement from a European plant that has since been abandoned our tests have been very satisfactory and substantiate Mr. Mattimore's conclusion No. 1 which has also been expressed by numerous other investigations.

The unsatisfactory behavior of the sample of European alumina cement mentioned above seems to have been due to the presence of carbides. We were unable to determine the precise quantity or nature of these carbides but did establish the presence of carbides which would be expected to reduce the quantity of cementitious bodies and thereby seriously depreciate the activity of the cement.

In the earliest use of alumina cement in this country, it was thought by many that excessive amounts of gaging water were required to satisfy the requirements of calcium aluminates for water during their setting processes. The writer was unable to accept this explanation of the need of alumina cements for water, as the maximum hydration of the calcium aluminates present would require much less than the amount of water needed to give a stiff consistency. Subsequent work as conducted by Mr. Mattimore and others has shown that the addition of water is needed not to form the hydrous calcium aluminates to which the rapid hardening of this cement is due but as a cooling medium to carry away the heat formed by the cement during the hardening process. It has also been found that the water for cooling purposes is most advantageously added at the beginning of the hardening period rather than in the form of excess gaging water when the concrete is mixed. Under field conditions where the exit of heat is restrained for any reason, we have found it vitally essential to apply water as a cooling medium from the fourth to the twentieth hour after placing of the concrete.

[1] President, E. L. Conwell and Co., Inc., Philadelphia.

Mr. Conwell.

The development of considerable heat by alumina cement during its hardening period seems to make it self protective against injury by freezing temperatures down to zero provided the concrete when first exposed shows a temperature of 70° F. or more. As would be expected, the need of water as a cooling medium decreases with lower temperatures and at extreme temperatures the absorption of heat by the air is found sufficient to prevent the accumulation of heat within the concrete.

We have made a number of analyses of top coats of alumina cement mortar some of which turned out satisfactorily while others failed to develop the usual strength and hardness. We found that the quantity of chemically combined water in the defective samples to be approximately one-half the quantity present in the samples which attained normal hardness. These results, while inconclusive, indicate that complete hydration of the calcium aluminates, to which the hardening of alumina cement is principally due, is prevented by the accumulation of heat in the mass with resultant high temperatures. As these calcium aluminates begin to lose their water of hydration at low temperatures, it is thought that the heat of setting if confined, is sufficient to partially dehydrate calcium aluminates.

Summarized, our conclusions are that the conventional methods of curing of road slabs or floor slabs by means of coverings such as burlap, sawdust or sand are not only useless but harmful, as these expedients act as insulators to prevent the exit of heat. It is felt that the surface should be left uncovered and that an amount of water appropriate to the existing temperature should be added by sprinkling to act as a cooling medium.

Among our tests of alumina cement, the following may be of interest as showing the continuous gain up to the one year period.

COMPRESSIVE STRENGTH, LB. PER SQ. IN.[a]

	1 Lumnite Cement 2 Sand 4 Pebbles	1 Lumnite Cement 3 Sand 6 Pebbles
1 day	3441.0	2071.0
2 days	3818.0	2444.0
3 days	4127.0	2598.0
7 days	4391.0	2663.0
28 days	4462.0	2691.0
3 months	4813.0	2748.0
6 months	4846.0	2962.0
9 months	5009.0	3317.0
1 year	5072.0	3460.0

[a] Each value represents the average of 3 specimens.

Mr. Miner.

MR. J. L. MINER[1] (*presented in written form*).—Mr. Mattimore's results are of more general interest than they would, perhaps, at first appear. The effect of the method of curing on the quality of the concrete, whether made of portland or alumina cement, is a subject worthy of further investigation. The effect of the method is more immediately noticeable with alumina cements because of their more rapid hydration.

It has been asked whether the application of water to exposed surfaces of concrete is necessary to replace the gaging water lost by evaporation or to supply by infiltration additional water required to carry on the hydration of the cement. It is very probable that this additional water is often required for both reasons.

Surface dusting as a result of drying out is an indication of the loss of gaging water by evaporation and probably also of the withdrawal of water from the surface into the concrete by the chemical action of hardening which, in the case of alumina cements, seems to start in the interior. The following test results are given to show the effect of supplying additional water during hydration:

1:2:4 alumina cement concrete molded in 3 by 6-in. cylinders and covered with glass plates and canvass until tested, developed at 24 hours a compressive strength of 1556 lb. per sq. in.

Concrete cylinders made of the same materials but stored at the end of 5 hours in a moist room where the top surfaces were exposed to moisture from a whirling spray, developed a compressive strength of 4384 lb. per sq. in. at 24 hours.

6 by 12-in. cylinders of 1:2:4 alumina cement concrete were covered with steel plates and wet bags as soon as made. At 24 hours they developed a compressive strength of 2990 lb. per sq. in.

One year later, what remained of this same lot of cement was retested, using similar aggregates in the same proportions and mixed at the same consistency. As soon as they were made the specimens were covered with glass plates. Five hours later the plates were removed and the top surfaces of the test specimens were sprinkled with water at frequent intervals until 18 hours old, when they were covered with wet bags until tested. At 24 hours these cylinders developed a compressive strength of 4290 lb. per sq. in.

The foregoing tests as well as observations in the field seem to confirm Mr. Mattimore's conclusions that any covering over the surface that tends to retain the heat developed by the concrete should be avoided. An exception to this, of course, would be when placing concrete in cold weather.

[1] 814 Second Place, Plainfield, N. J.

Mr. Miner.

Mr. Mattimore has pointed out that there appears to be a critical period when water should be applied to exposed surfaces. If water is applied before this time, exposed surfaces will dust, for the same reason that exposed surfaces will dust if the cement is mixed with an excessive amount of gaging water. If water is not applied reasonably soon after this critical period, the full strength of the cement will not be developed and exposed surfaces will dry out and dust. In this respect alumina cements differ from portland cements only as to when the water should be applied.

Mr. J. Bertet in his report on the "Influence of the Quantity of Mixing Water on the Heating During the Hardening of Aluminous Cements"[1] noted that there appeared to be a break in the rise in temperature at about the sixth hour, indicating perhaps the end of one step and the beginning of another in the process of hydration. It is interesting to note that this break occurs at about the time Mr. Mattimore suggests that water be first applied to the surface of alumina-cement concrete.

Mr. Merriman.

MR. THADDEUS MERRIMAN.[2]—What is true for the aluminous cements as pointed out by Mr. Mattimore in his excellent paper and by Mr. Miner in his discussion, is true for portland cement also. A few moments ago Mr. N. C. Johnson drew on the board a cross-section of a highway, and indicated that if there be evaporation from the surface, moisture will travel toward that surface, and that there will be a concentration of the solubles along the plane of evaporation. Now, if there were no evaporation from the surface, the solution concentration, within which the reactions of hydration occur, would, even so, not remain constant because water is continually being withdrawn from the free water present and bound up as water of hydration or water of crystallization, as the case may be. If, therefore, a film of water be supplied on the surface of the concrete sufficient in quantity to compensate for the loss by evaporation and at the same time permit water from the film to pass down into the mass and so hold the solution concentration constant, a condition will result in which the hydrations will proceed to best advantage. On the other hand, if more water than sufficient to accomplish this end be put on top of the concrete as, for instance, in the ponding method, the reverse condition obtains. The solubles from the more concentrated solution will then tend to pass upward into the water of the pond in the effort to equalize the concentrations as between the solution within the mass and the water solution on top. It is easy to see, therefore, that there is a point

[1] *The Structural Engineer*, July, 1924.

[2] Chief Engineer, Board of Water Supply, City of New York.

Mr. Merriman. beyond which ponding should not be carried, that is, the quantity of water in the pond should be held down to a minimum unless, indeed, the solution concentration of the water within the pond is controlled by putting into it salts of the same kind and nature as are found within the mass of the cement itself. This matter of the curing of concrete which, up to date, has received little if any consideration, is after all one of the very important items which we must consider.

Mr. Freeman. MR. P. J. FREEMAN.[1]—Last year in the fall and late winter the Allegheny County Department of Public Works used about 14 carloads of high-alumina cement. We did not do very much laboratory work on it. We simply saw a place where we thought it worth while to take a chance and use it on the representation of the manufacturers, which we did. In one case the cement was used in replacing a fairly good sized culvert that had suddenly washed out on a very busy road, and in other cases on some of our other roads that unfortunately had not enough foundations on the hillsides and had partly slipped away. Reinforced slabs were put in, and various patches and similar work of that kind. No new road slabs were put in but in some cases there were sections 10 ft. wide and 50 or 100 ft. long.

Referring to Mr. Mattimore's conclusions, for the first, we obtained a higher strength at 24 hours than we were getting under our routine testing with a portland cement at 28 days, averaging from 4000 to 5000 lb. per sq. in. on a 1:2:3 mix. We did not cover the section with burlap. First, we did not think about it because they were not very large sections, and then we did not have the burlap, so we avoided that difficulty.

Jumping down to item 4, we started putting the water on at just about the seven-hour period, being governed by the fact that if we turned the hose on it, it would mutilate the surface, so whenever it got to be so we could throw water on it without hurting the concrete, we proceeded to do it. On every job the watchman was very carefully instructed; I personally instructed the foreman and the superintendent and then went around and instructed the watchman, and usually had somebody check up on him in addition to that. I drove over most of this work within the past week or ten days, and so far as I can tell it is all in a very satisfactory condition, although none of it was put in under special supervision except as to the curing. We simply turned it over to our maintenance department—or, in one case that was a contract job, it was handled just as any portland cement job would be handled. In fact, on one job which involved a forty-thousand-dollar contract, we used portland cement on about half of

[1] Chief Engineer, Bureau of Tests and Specifications, Department of Public Works, Pittsburgh, Pa.

the work because of the cheapness. We put high-alumina cement on one side and opened it to traffic, and then used portland cement to finish the job. No special considerations, other than these mentioned, were used, and we did not observe any unusual shrinkage of the patches—in fact, I think they were probably in better condition than our portland-cement patches. Mr. Freeman.

MR. H. S. MATTIMORE.[1]—In closing I wish to especially emphasize that it is important to apply water to the surface of alumina cement concrete in large quantities, at what we have found to be the critical period, that is, when the surface shows a drying out appearance. Where practical to do so, the ponding method seems to be the most positive one, in that there is an assurance of sufficient water being added. This may be somewhat in conflict with Mr. Merriman's theory, in that with the ponding method there would be several inches of water on the surface, but in our experimental work this method results in a hard surface. Mr. Mattimore.

[1] Engineer of Materials, Pennsylvania State Highway Department.

A NEW TEST FOR CONSISTENCY OF PAVING CONCRETE

By F. H. Jackson[1] and George Werner[2]

Synopsis

This paper presents the results of a series of tests conducted by the U. S. Bureau of Public Roads for the purpose of determining the value of a new test for consistency of paving concrete.

The process is based upon the principle that the consistency of concrete may be determined by weighing the amount retained upon a circular plate of given diameter when the concrete is deposited thereon in a standard manner. The test results indicate that for any given mix the relation between consistency, as determined by this method, and water-cement ratio is essentially the same as the strength - water-cement ratio relation, indicating the possibility of using this test as a direct measure of the probable relative strength of the concrete. The test results likewise indicate that variations in the size and type of aggregate do not affect the results to the same extent as in the case of either the slump or flow test.

The development of a simple yet reliable method for determining the consistency of cement-concrete mixtures presents a problem that has never been satisfactorily solved. Although numerous schemes have been proposed from time to time, only two or three of them have ever been developed so as to be of any practical value. The so-called slump test[3] has been used more extensively than any other method and, when properly interpreted, has given fairly good results especially as a method of field control. Its obvious limitations, however, especially as regards its use in the control of dry mixtures, lean mixtures, and concretes in which large sized crushed stone is used, have prevented its general adoption. The flow test[4] has also been tried out by a number of investigators and is used to a certain extent as a method of laboratory control. The nature of the apparatus required, however, makes the flow test impractical to use as a field method. Mention should also be made of the penetration test for "workability" of concrete proposed by Pearson and Hitch-

[1] Engineer of Tests, U. S. Bureau of Public Roads, Washington D. C.

[2] Assistant Scientific Aid, U. S. Bureau of Public Roads, Washington, D. C.

[3] Tentative Method of Test for Consistency of Portland-Cement Concrete for Pavements or for Pavement Base (Serial Designation: D 138 – 22 T), *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part I, p. 805 (1922); see also *Proceedings*, Am. Soc. Testing Mats., Vol. 25, Part I, p. 746 (1925).

[4] W. L. Schwalbe, "A Comparison of the Results of the Slump Test and the Flow Table in the Measurement of the Consistency of Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 21, p. 983 (1921).

cock.[1] This method would appear to be limited in application to concrete of the consistency usually required in reinforced concrete building work rather than to the dry mixes employed in road construction.

There has been a great deal of discussion before this Society and elsewhere during the last four or five years regarding just what is meant by the terms "consistency," "workability," etc., as applied to concrete mixtures. According to Abrams,[2] consistency may be defined as "the relative plasticity or workability of freshly mixed material." Pearson and Hitchcock, on the other hand, in their paper describing the penetration test, distinguish between consistency and workability, holding that the former term should be used only to describe the condition of the concrete as it is affected by changes in water content, whereas the latter term should be used to describe that condition of a given mixture which depends not only upon the water content, but upon any factor which affects the amount of work required to place and finish the concrete in a satisfactory manner. It is, of course, apparent to anyone that there is a great difference in the "workability" of a rich, high-sanded 1:2:3 mix as compared with a lean 1:3:6 mix, even though both mixes may have the same "consistency" as measured by the slump or flow test. Furthermore, it is impossible by any change in the water content of the leaner mixes to secure as workable a mix as may be obtained with the former. From this viewpoint, therefore, it would appear that there is justification for a distinction between the terms. It has often been demonstrated, however, that by far the greatest variation in the strength of concrete in actual practical construction is due to changes in the water content. Granting, therefore, the effect of water on strength, and bearing in mind also that variations in the amount of water required to produce concrete of a given consistency may be caused by any of the factors which affect workability, such as gradation of aggegrate, cement content, etc., it would appear that the practical control of the water is the vital thing to be considered. In this paper a new test is described which may be used for controlling the water content and therefore the strength of any given concrete mixture.

The test was designed as a substitute for the slump test, and is proposed for use primarily as a method of field control. The process is based upon the principle that the consistency of concrete may be

[1] J. C. Pearson and F. A. Hitchcock, "A Penetration Test for the Workability of Concrete Mixtures with Particular Reference to the Effects of Certain Powdered Admixtures," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 276 (1923).

[2] D. A. Abrams, "Importance of Consistency of Mix in Concrete Work," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part II, p. 443 (1923).

determined by weighing the amount which is retained upon a circular plate of given diameter when the concrete is deposited thereon in any standard manner. The device which has been used by the Bureau of Public Roads in demonstrating this principle is shown in Fig. 1.

The apparatus consists essentially of a box spring scale upon which is mounted a circular steel plate 12 in. in diameter. A hopper

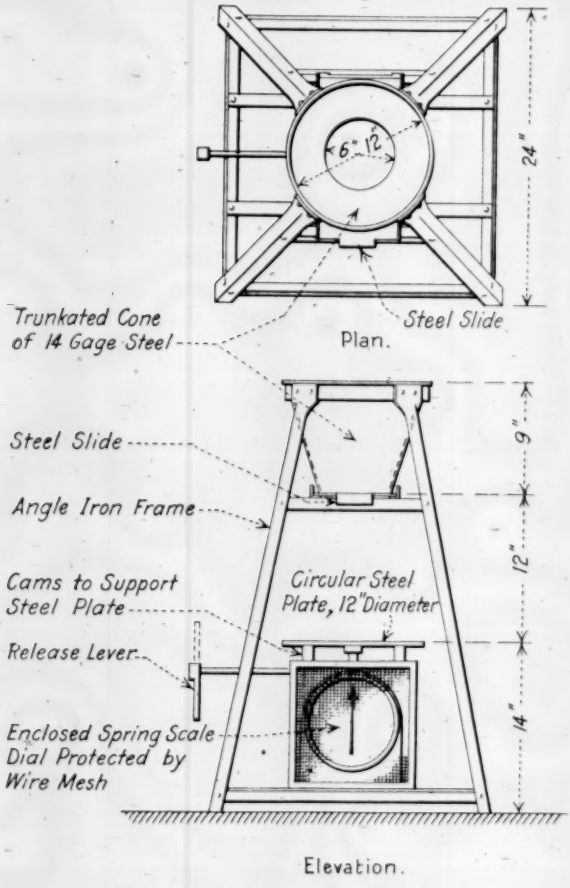

FIG. 1.—Consistency Apparatus.

supported by an angle iron frame is mounted over the plate, at such a height that the point of discharge is 12 in. from the surface of the steel plate. The hopper is of such capacity as to hold approximately 45 lb. of wet concrete, and is provided at the discharge end with a steel slide. The test is made in the following manner. The apparatus is set up adjacent to the point where the batch of concrete that is to be tested will be deposited, and the hopper filled with

concrete directly from the pile by means of shovels. It has been found that the exact procedure to be followed in filling the hopper is immaterial so long as it is filled completely and no attempt is made to

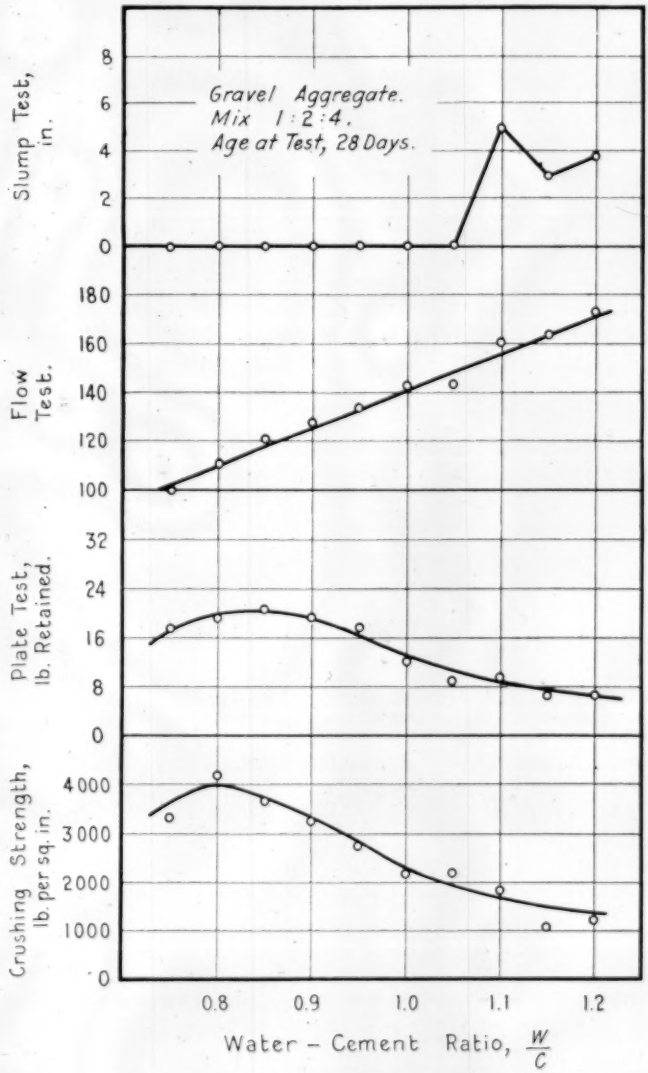

FIG. 2.—Effect of Water on Strength and Consistency of Concrete (First Series).

jostle or compact the concrete. Mounted underneath the steel plate are two cams operated by a handle which are so arranged that the weight on the plate may be taken off the spring. With the cams

set in this position the slide is drawn and the concrete allowed to flow out upon the plate until the hopper is empty. For a 12-in. plate, the hopper is of such capacity that it will hold about 50 per cent more concrete than it is possible to retain upon the plate under any circumstances. After all movement has ceased, the handle controlling the cam is turned and the concrete upon the plate weighed.

Numerous tests with this device on concrete of various proportions and with various sizes and types of aggregate indicate that it will truly measure the relative consistency of the mix. In one series of tests, for instance, in which gravel graded up to 1½ in. in size was used in a 1:2:4 mix, the amount of concrete retained upon the plate varied from 21 lb. with a water-cement ratio of 0.85 to 6 lb. with a water-cement ratio of 1.2. In this series of tests which was made some months ago a number of batches of 1:2:4 concrete were made up in which the water-cement ratio was increased successively by 0.05. Slump tests, flow tests, and tests with the 12-in. plate were made on each batch after which the material was cast into 6 by 12-in. cylinders and tested for compressive strength at the age of 28 days. Results of this series of tests are shown in Fig. 2. Note the striking similarity between the curve showing the relation between water-cement ratio and strength and the curve for water-cement ratio and consistency as determined by the plate tester. It will be observed that there is a tendency for the amount of concrete retained upon the plate to drop off for very dry consistencies in exactly the same way as the strength falls off. It occurred to the authors that if the relationship shown in Fig. 2 were true throughout the entire range of working consistencies and for various mixes and types of aggregate that the plate test should give directly a very good indication of the probable relative strength of the concrete. In this particular series the slump test showed up very poorly, although it was made strictly in accordance with the procedure described in the Society's Tentative Method of Test for Consistency of Concrete (D 138 - 22 T), except that the mold was removed immediately after placing the concrete. It will be noted that the concrete did not slump at all until a water-cement ratio of 1.1 had been reached, corresponding to a flow of 160 and a plate test of 9 lb. Granted that this is very unusual, these results nevertheless serve to illustrate the uncertainties of the slump test. The flow test results in this series were, on the other hand, very concordant, showing practically a straight line relation for a water-cement ratio varying from 0.75 to 1.2.

Having in mind the interesting possibilities in the way of strength control as indicated by this initial series of tests, a somewhat more

extensive investigation was begun for the purpose of determining whether these relations held for different types and sizes of aggregates and different proportions. In this series of tests, six different mixes were used: 1:1½:3, 1:2:3, 1:2:3½, 1:2:4, 1:2½:5, and 1:3:6. Three types of aggregates—Potomac River gravel, crushed limestone, and crushed slag—were employed for each mix. The gravel and slag were graded from ¼ to 1½ in. and the limestone from ¼ to 1½ in. for one lot and ¼ to 2½ in. for another lot. The concrete was mixed by hand with shovels in 1-cu-ft. batches. Three consistencies were

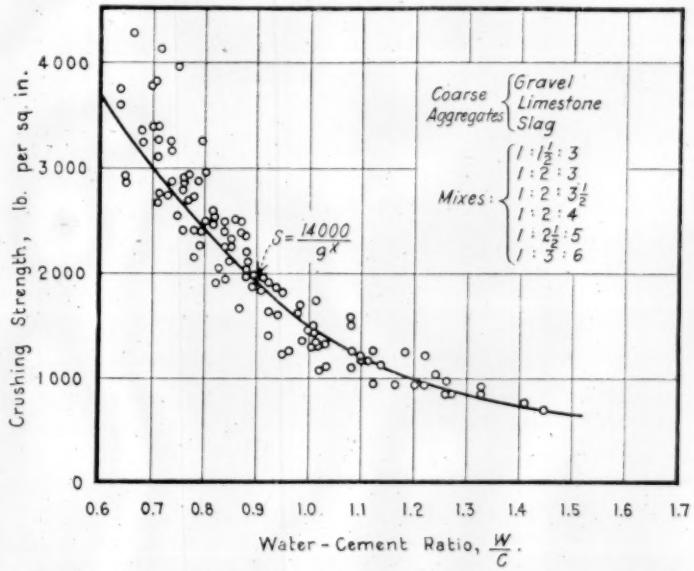

FIG. 3.—Relation Between Water Cement Ratio and Crushing Strength of Concrete.

used for each combination—dry, medium, and wet. Four 6 by 12-in. compression test specimens were made from the concrete in each batch. Immediately after mixing, the three tests for consistency—plate test, flow test, and slump test—were made simultaneously on different portions of the batch, by different operators, in order to eliminate any error due to the time element or to rehandling the concrete. At the conclusion of the first series the entire program was repeated, making a total of approximately 130 separate batches of concrete tested. In order to check up on the accuracy of the strength - water-cement ratio relation, the results of all of the compression tests, irrespective of mix, size or type of aggregate were plotted, and are shown in Fig. 3, together with the curve suggested by Abrams and

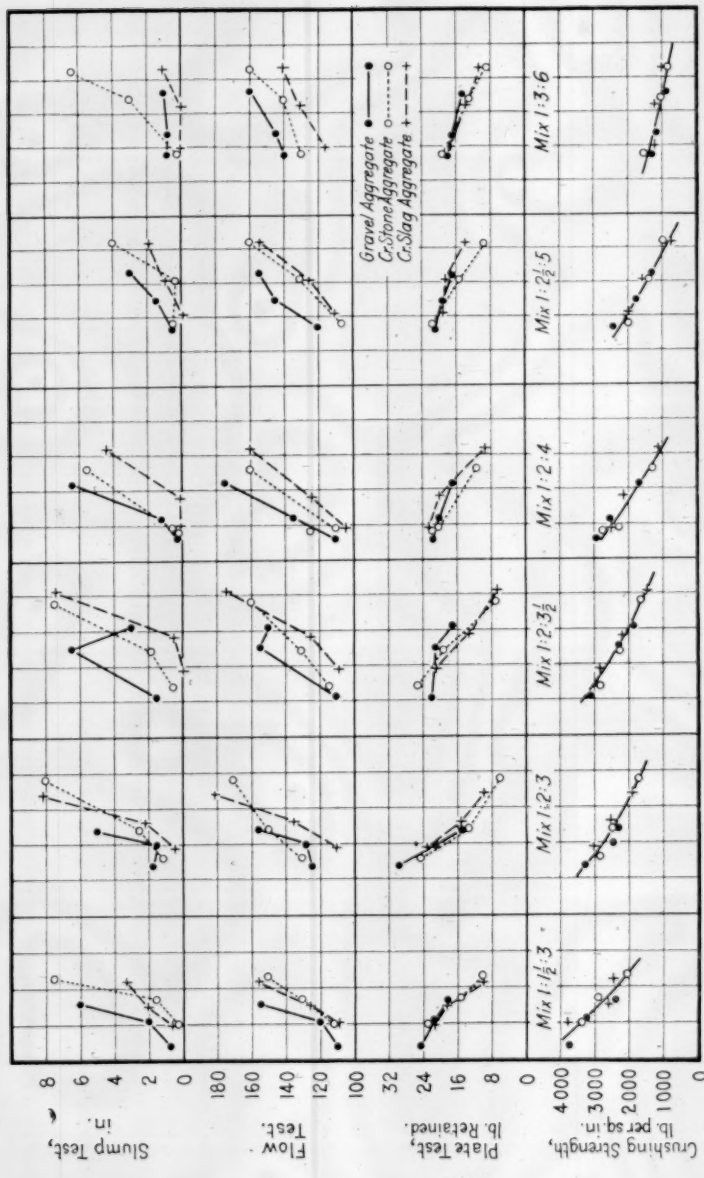

FIG. 4.—Effect of Water on Strength and Consistency of Concrete (Second Series).

Walker[1] as indicating the strength - water-cement ratio relation. The equation of this curve is $S = \frac{14{,}000}{9^x}$ in which S equals the crushing strength of the concrete in pounds per square inch at 28 days, and x (an exponent) is the water-cement ratio. It will be seen that the averages of all points lie very close to the curve except for the very dry mixes, although it should be noted that these tests were made at the age of 14 instead of 28 days, so that the average of all of the points would probably lie somewhat higher than is here indicated if they had been made at the conventional period. In order to determine relations between slump, flow, plate test, and strength for the different mixes and types of aggregate, the results were plotted separately on a series of charts which were reproduced as Fig. 4. It will be observed that the same general relation between strength and consistency as determined by the plate test is indicated here as is shown in Fig. 2. Furthermore, the points representing consistency of concrete with the different coarse aggregates appear with one or two exceptions to lie very much closer together than in the case of either the slump or flow test, indicating that determinations of consistency by this method are more independent of the type of aggregate than in the case of either of the other methods. In view of the fact that the compressive strength of the concrete appears also, in general, to be independent of the type of aggregate, this would appear to be an advantage. Note also the fact that the slope of the average curve for consistency as determined by the plate test appears to parallel in general the curve showing the relation between water-cement ratio and strength. It will be seen, for instance, that for the leaner mixes, such as 1:2½:5 and 1:3:6, the curves are much flatter than in the case of the richer mixes. The 1:2:3 mix shows the steepest slope and likewise the greatest range in consistency, which is to be expected in view of the high plasticity or workability of a mix of this nature. When we observe the results of the flow and slump tests, however, we find that the points representing the various types of aggregates are much more widely separated. In the slump test, for instance, it will be observed that for a given water-cement ratio the points representing the tests with the gravel aggregate lie in general considerably higher than in the case of either the stone or slag concrete, while the stone in general is next and the slag lowest. These results are in accord with general experience with this test as a measure of

[1] Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill., *Bulletin 9*, 2d Ed., April, 1925.

consistency. The slump test appears likewise to be much more erratic for leaner mixes than in the case of the richer mixes. This is also borne out by experience. The flow test results are fairly concordant for each type of aggregate. They, however, show the same general separation by groups as is indicated in the slump test.

For the purpose of checking up on the value of this device as a method of field control, a series of tests was run in connection with a concrete paving project. On this project rather poorly graded crushed limestone was being used as coarse aggregate, with a consequent tendency toward segregation whenever too much water was used. By the use of this device it was possible to control the mix so that a considerably drier consistency was obtained with consequent freedom from segregation, and undoubted increase in strength.

CONCLUSION

In conclusion, the authors feel that enough work has been done with the plate device to warrant its presentation as a possible method of field control for the consistency of concrete. Extreme accuracy is not claimed. In no case is it possible to check closer than 1 or 2 lb. on the plate. However, the data which have been obtained indicate that this is sufficient accuracy for all practical purposes. The fact that the test gives practically the same results irrespective of the type of aggregate would appear to be a point in its favor, as would also the fact that, for a given mix, it appears to be a direct measure of the crushing strength of the concrete.

Acknowledgment.—The authors wish to take this opportunity of acknowledging the assistance of Mr. A. A. Levison, Chief Engineer of the Road Department of the Blaw-Knox Co., and formerly with the Bureau of Public Roads, in connection with the early development of this method of determining the consistency of concrete.

DISCUSSION.

MR. W. A. SLATER.[1]—This paper is of interest not merely in presenting a new method of measuring the consistency or workability, or whatever you will, of the concrete, but it is of a great deal of interest in showing what the function of such a measure is. I can remember about two years ago undertaking to answer before a question box discussion of the American Concrete Institute how to make use of the slump test in the field. When I tried to answer the questions, I found I had a good sized job on my hands. Since then in the field I found with the slump test that there were discordant results from one time to another. What it meant it was hard to say. In the laboratory the slump test has given fair results. Even in the field the results of the slump test have been consistent with those of the flow test, but there has been little or no relation between slump and strength. This paper shows that too little aggregate affects the workability in much the same way as too much water, and yet its effect on the strength of the concrete is opposite to that of too much water. There is likely to be as much uncertainty in the measurement of the aggregate as of the water and this will indicate why results of field tests show no relation between slump and strength. If too large a slump may be due to either too much water or too little aggregate, it is apparent that its function is merely to indicate that a checking up of the measurement of materials is necessary. **Mr. Slater.**

The field test made at Camden, N. J., in connection with the report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, resulted in a lot of slump tests that, when plotted against the flow tests, were very consistent but which bore no relation to the strength of the concrete. We were compelled in the discussion of the slump test to say it did not seem to show very much, and yet we were not ready to recommend its abandonment. I think I could answer a little more confidently now why we should continue this test. I believe we can count upon it to tell whether anything is wrong in the measurement of materials but not to tell just what is wrong.

I should like to ask a question regarding the paper under discussion. One of the most difficult things we have to contend with in the slump test is that two successive measurements are likely to differ

[1] Engineer-Physicist, U. S. Bureau of Standards.

Mr. Slater. widely, even though the concrete is the same in both cases, and it is impracticable to make slump tests frequently enough to have several values to average at a given time. Now, if the tests that are averaged vary enough among themselves that the same difficulty would be present, I think it is unfortunate. If the agreement of individual tests was good, I think it is an important thing in its favor. I can not quite understand why it would not be possible to make a more accurate weighing than 1 or 2 lb., if greater accuracy is desirable. It seems to me, so far as the weighing is concerned, that it could be done. Possibly Mr. Jackson means that 1 or 2 lb. is about the degree of precision with which a test can be repeated.

Mr. Jackson. MR. F. H. JACKSON.—I would say that from the result of our work to date 2 lb. is the limit of variation between individual results. Usually we check a great deal closer than that. Two pounds on a basis of a 20-lb. weight is a 10 per cent variation, and it has been my experience that the percentage of variation in the slump test, particularly with broken stone aggregate, is considerably greater than that.

Mr. Gilkey. MR. H. J. GILKEY.[1]—I should like to know how accurately you must know the amount of concrete that is in the hopper above the scale.

Mr. Jackson. MR. JACKSON.—It is only necessary to fill the hopper completely. In other words, have enough concrete in it to allow some to run off the plate after the entire amount which can be retained has piled up on it. It is not necessary to weigh the concrete that goes into the hopper at all. We have found it, however, extremely important not to disturb the concrete in any way after it is placed in the hopper; to do so will vitiate the results to a considerable extent.

Mr. Ashton. MR. ERNEST ASHTON.[2]—I should like to ask what consideration was given to the height of the drop.

Mr. Jackson. MR. JACKSON.—So far we have used a constant height. The height of the drop was the same as the diameter of the plate. We experimented with 15 and 18-in. diameter plates, and found that with large aggregates up to 3 or $2\frac{1}{2}$-in. size, we got slightly more concordant results with the 15-in. plate, but that with $1\frac{1}{2}$-in. aggregates we got more concordant results with the 12-in. plate.

Mr. Ingberg. MR. S. H. INGBERG.[3]—What was the time after the aggregate was on when you made the weighing?

Mr. Jackson. MR. JACKSON.—The weighing is made immediately after the concrete upon the plate comes to rest. The concrete is simply shovelled into the hopper with an ordinary shovel until it piles up, when the slide

[1] Associate Professor of Civil Engineering, University of Colorado, Boulder, Colo.
[2] Chemical Engineer, Lehigh Portland Cement Co., Allentown, Pa.
[3] Physicist, U. S. Bureau of Standards.

is withdrawn and the concrete flows out upon the plate exactly as it would flow out on a subgrade when deposited from a bucket. Immediately after all movement has ceased we take the weighing. **Mr. Jackson.**

MR. A. A. LEVISON.[1]—The point that I think has not been brought out in connection with this new test described by Mr. Jackson is the practicability of making the test—if it should develop subsequently that it can be applied to construction concrete—job concrete. The apparatus is easily portable. The concrete requires no manipulation of any kind after it has been mixed, and there is no puddling required. It appears that the test is one that can be repeated more rapidly and a good many more times with greater ease than any other test for consistency. **Mr. Levison.**

There are certain things in connection with any test for consistency that I should like to mention. There are certain recognized factors that have a definite effect upon the consistency of the mixed concrete. One of them is the grading of the coarse aggregate for the mix. We know that on the job we experience considerable trouble due to the variation in the grading of the coarse aggregate, and it would appear that any scheme for testing the consistency of concrete would need to take that variable into consideration.

There has recently been developed a method which has already attained considerable use for controlling the water-cement ratio in concrete quite accurately. This method is known to most of you; it is termed the inundation method. It has the advantage over any test for consistency in pre-determining the amount of water that will enter the batch of concrete. It would seem as though a method of that kind properly applied to the making of concrete really solves the situation with regard to consistency. We find in practice that although the water-cement ratio is controlled effectively by means of the inundation method, the consistency is not as constant or as uniform as one might expect, and we know that the variation in the grading of the coarse aggregate has a great deal to do with the variation in the consistency even though the water-cement ratio is maintained uniformly constant. It is our belief that the matter of controlling the grading of the coarse aggregate for concrete is deserving of the very best consideration and study.

There is another point that I should like to say something about. Mr. Jackson in his discussion tried to differentiate between the terms consistency and workability. It struck me that that would be a mighty difficult thing to do on the construction job, because if consistency and workability cannot be correlated in some way, the measuring of con-

[1] Chief Engineer, Road Department, Blaw-Knox Co., Pittsburgh, Pa.

Mr. Levison. sistency loses its value to a large extent. The practical consideration of consistency has a great deal to do with workability. The job is laid out with a certain scheme for distributing and putting the concrete in place. To a large extent, that scheme for distributing and placing the concrete together with the kind of construction being built determines the consistency or the workability of the concrete, whichever term applies.

Mr. Wig. MR. R. J. WIG.[1]—There is one question I should like to ask Mr. Jackson. In making use of this apparatus would it be necessary to write a separate specification for every group of aggregates on account of variation in weight of aggregates or could you establish one specification that would apply to all aggregates?

Mr. Jackson. MR. JACKSON.—I think that we could possibly establish one specification that would apply to various types of aggregates, but I am not so sure yet whether we could establish one specification which would apply to various sizes of aggregates. I think that the difference in the weight of the aggregate itself is relatively small, compared to the total weight of the concrete on the plate. As you possibly observed from Fig. 4, the results with the slag were remarkably close to the results with limestone and gravel, and yet the slag weighed only about 75 lb. per cu. ft., whereas the limestone weighed about 90 lb. per cu. ft.

Mr. Herschel. MR. W. H. HERSCHEL.[2]—I do not want to pose as an expert on concrete, but I do know something about what I call consistency. At any rate, the essential feature of consistency of a plastic material—and I think, according to certain definitions of plasticity, you must admit that concrete is plastic—is that the ratio between the rate of flow and the force producing the flow is different at low than at high rates of flow. In a viscous liquid, for example, there is a constant ratio between flow and pressure and we can express the consistency of a viscous liquid, that is, its viscosity, by a single numerical value. But when we come to a plastic material, such as concrete, it is utterly impossible to express the complete relation between flow and pressure by any single numerical value, no matter how obtained. I believe that that is one of the fundamental difficulties in this situation, that you are trying to express the consistency of a plastic material by a single numerical value, a thing which I believe to be utterly impossible.

Mr. Slater. MR. SLATER.—I want to ask if Mr. Herschel will not tell us what to do. He told us what is wrong with the situation. I wonder if he can not tell us what to do.

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[1] Vice-President, Celite Products Co., Los Angeles, Calif.

[2] Associate Physicist, U. S. Bureau of Standards.

MR. HERSCHEL.—As I have said, the ratio between the rate of flow and the pressure which produces the flow is not constant for plastic material. If you once admit concrete is plastic then it necessarily follows that the only way to get the complete relation between flow and pressure is to make successive observations at different rates of flow. That is the essential requirement in testing the consistency of a plastic material. It is a well known procedure in testing of paint, for example, as developed by Bingham, and there are a great many other collodial materials that appear to be true liquids so far as observation goes, but when tested at varying rates of flow, they prove to be plastic. We have tested benzene solutions of rubber, and find they are plastic even at 0.2 per cent concentration. That is the fundamental principle, that you must make successive observations at different rates of flow. The details of that general principle must be worked out according to the nature of the material tested. Mr. Herschel.

THE SIGNIFICANCE OF THE COMMON TEST METHODS FOR DETERMINING THE STRENGTH OF MORTARS[1]

By John W. Gowen[2] and H. Walter Leavitt[3]

Synopsis

Differences of opinion have recently arisen as to the relative value of the standard tension and compression tests upon sand mortars. It is the purpose of this paper to outline some of the chief functions of tests in general and to establish the significance of the standard tension test, the standard compression test, and a new abrasion test upon portland-cement mortars.

The general functions or requirements of any test may be cited as follows:

1. It must give a quantitative measure of the most important stresses to which the material will be subjected in the structure where it is to be used.
2. It must give concordant results when the tests are repeated on the same material.
3. It must give new information and not duplicate the facts established by other tests.

The method of analysis used relates to that field of the Calculus of Observations which is commonly referred to as the "Correlation Method." The paper describes the application of this method to a study of the three above-mentioned tests in relation to the three general functions named.

The method and its use, the authors believe, opens a new and interesting field of analysis of test data upon engineering materials.

If the purpose of laboratory testing of structural materials is considered broadly, the justification for the use of any test for strength depends upon the supposed fact that it gives a quantitative measure of the strength characteristics of the material as they will be found in actual practical construction in the field. It may be impossible to design a single test to meet these field requirements. In this event other test methods should be developed. These secondary tests are justifiable for use so long as they show different strength characteristics of the material. In other words, any tests applied to structural materials should develop new facts about the strength of those materials and not simply duplicate the information obtained from some other previous test. The authors have yet to see a demonstration of such a justification for the tests now in use in practical mortar testing. It is the purpose of this paper to present data to

[1] Contribution from Maine Technology Experiment Station.

[2] Biologist, Maine Agricultural Experiment Station, Orono, Maine.

[3] Testing Engineer, Maine State Highway Commission, and Associate Professor of Civil Engineering, University of Maine, Orono, Maine.

show what qualifications the present tension and compression tests on mortar have for meeting this need.

Perhaps the first requirement that a test must meet if it is to be worthy of consideration is that it should be designed to measure the most important stresses to which the material will be subjected when that material is used in construction. The field requirements should be measured by laboratory tests in so far as it is possible. This condition is generally accepted as axiomatic by all engineers. The second requirement follows directly from the first, namely, if the test is properly designed it must give concordant results when repeated on the same material. In other words, there must be good correlation between the performance of one test specimen and that of another test specimen when they are made identically, both as to procedure and as to materials, making it possible to check the results quite closely. For the third requirement, the test must give information about the strength of the material which is not given equally well by other tests. In other words, the test should have little or no relation with other tests designed for measuring the strength-giving qualities of the material under study.

These facts show the need for careful study of all the tests now in common use. This need is also shown in the conflict which has arisen in the minds of testing engineers concerning the relative merits of the tension and compression tests on mortars. This difference of opinion has led certain engineers to adopt only one of these tests. However, if we considered the problem analytically, it would appear as if the tension and compression tests as now performed meet the first requirement, namely, that the tension test is designed to measure the resistance to pulling stresses and the compression test to measure crushing stresses. Both of these stresses are encountered to some degree in nearly all concrete structures.

When these tests are examined for the second requirement,—whether or not a given material in either of these tests will give consistent results from one specimen to the other,—it is also found that both of these methods of analyzing strength in mortars meet this requirement. This relationship may be expressed as a coefficient of correlation[1] between one test specimen and another test specimen when these specimens are made from the same mortar. This corre-

[1] The methods of statistical analysis of data involving correlation are described in the following texts:

G. Udney Yule, "Introduction to the Theory of Statistics," J. B. Lippincott Co., Philadelphia.
A. L. Bowley, "Elements of Statistics," P. S. King and Son, London.
D. Caradog Jones, "A First Course in Statistics," G. Bell and Sons, Ltd., London.
T. L. Kelley, "Statistical Method," MacMillan, New York.
E. T. Whittaker and G. Robinson, "Calculus of Observations," D. Van Nostrand Co., New York.

lation coefficient is measured on a scale of −1.0 to 0.0 to +1.0. If the correlation coefficient is +1.0 or nearly that, say +0.8, the individual breaking strength of one test specimen is shown to be nearly the same as the individual breaking strength of another test specimen. If on the other hand the correlation coefficient is 0.0

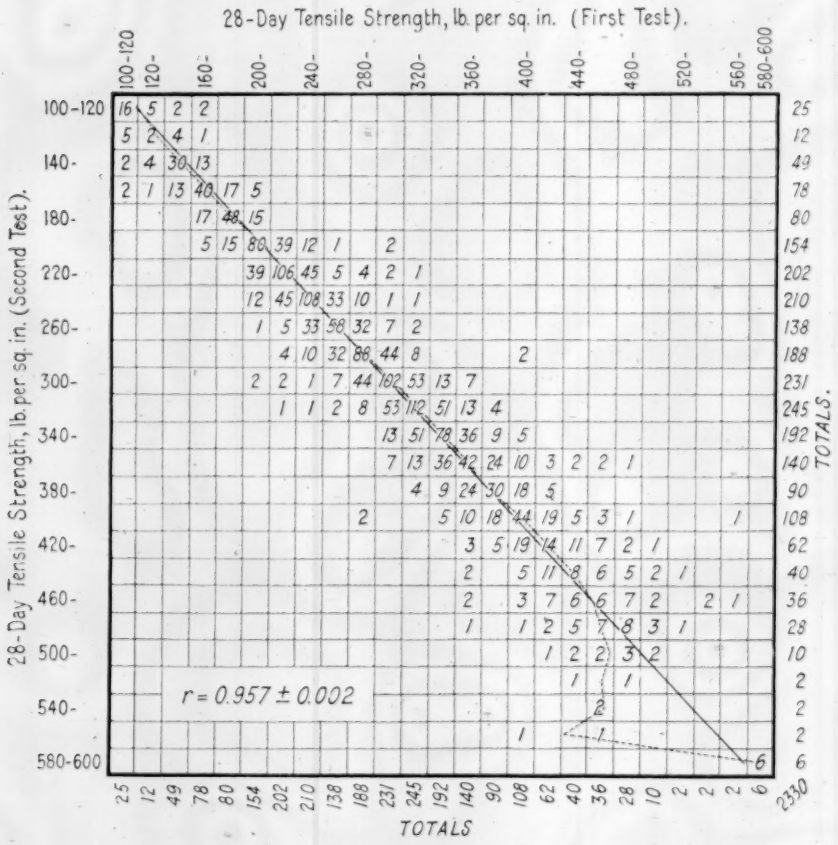

FIG. 1.—Tension Tests of Mortar, 28 Days (1:3 Mix, 391 Sands).

The tensile strength in pounds per square inch of the first tests are plotted as abscissas and the second tests as the ordinates. The value of the correlation coefficient r measures the interrelationship of the first tests with the second tests. The straight diagonal line is the "mean regression line" and the dash line is the "raw regression line." The whole chart is symmetrical, about a 45-deg. diagonal axis line, as will be seen by looking at the "total" columns. If a perfect correlation (1.0) existed, all the figures would fall in the squares cut by this 45-deg. line and the mean regression line would coincide with it.

there is no relation between the breaking strength of one test specimen and another; for example, a pair of tests where one has a high strength is as liable to have the other member of that series develop a low result as to have it give a high strength. The minus correla-

tions need not be considered here as they do not enter into this problem. They simply mean an inverse relationship.

ANALYSIS OF TENSION AND COMPRESSION TESTS

This coefficient as determined for the 28-day tension test upon the natural sands of Maine was found to be $+0.957 \pm 0.002$. Fig. 1

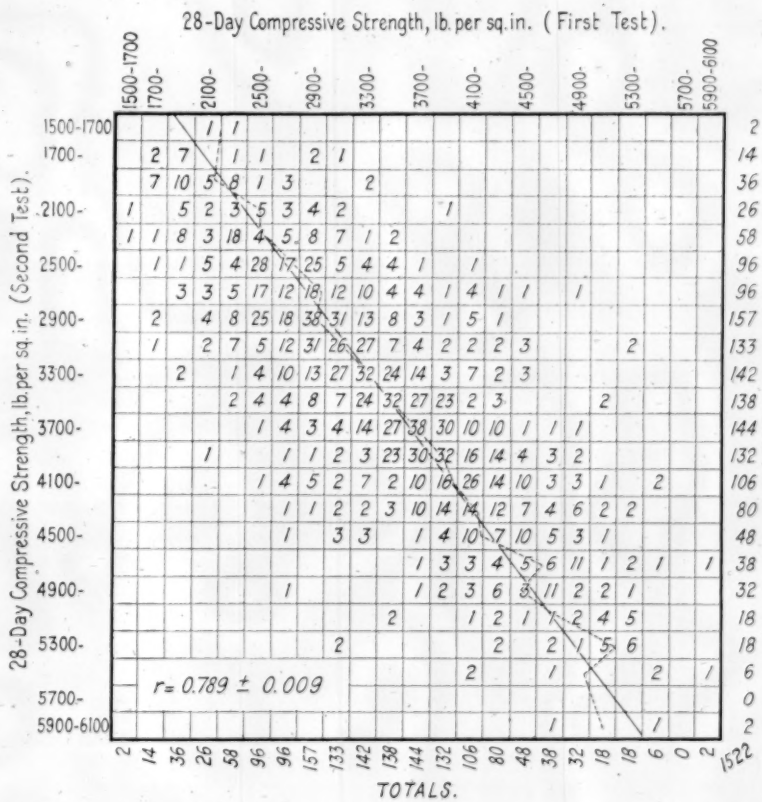

FIG. 2.—Compression Tests of Mortar, 28 Days (1:2 Mix, 255 Sands).

The compressive strength in pounds per square inch of the first tests are plotted as abscissas and the second tests as ordinates. The value of the correlation coefficient, r, measures the interrelationship of the first tests with the second tests upon each of the 255 sands. The mean and raw regression lines are represented as in Fig. 1.

illustrates this interrelationship of one briquette with another, 391 sands being considered.

The corresponding relationship for the 28-day compression test on 2 by 4-in. cylinders was found to be $+0.789 \pm 0.009$. Fig. 2 illustrates the interrelationship of one cylinder with another, 255 sands

(1:2 mix)[1] being involved in this analysis. These results show clearly that both the tension and compression tests meet the second require-

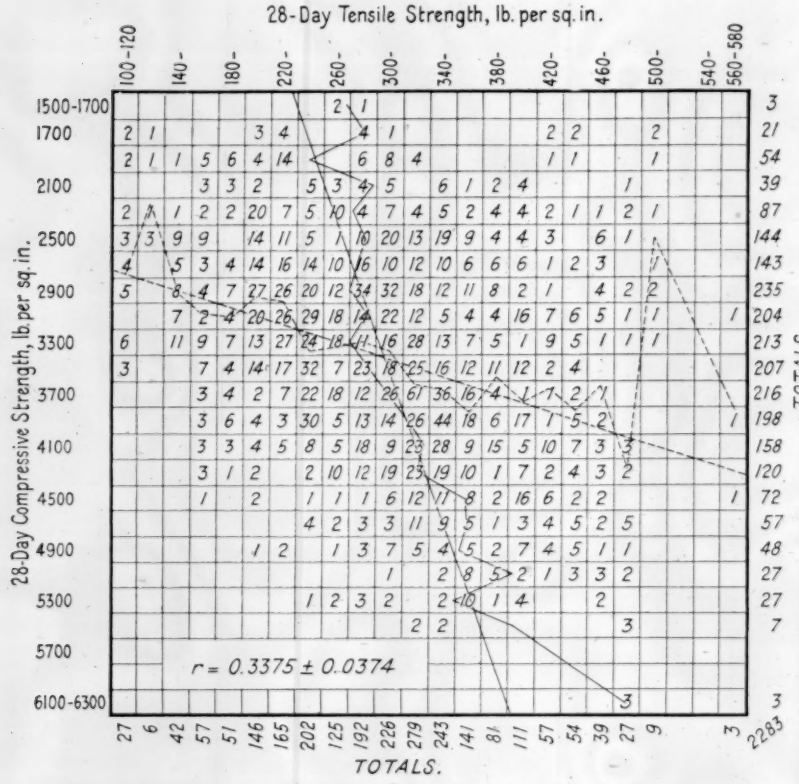

FIG. 3.—"Scatter" Diagram Showing Relationship between Tension and Compression Tests of Mortar.

The tensile strengths of the individual briquettes are plotted as abscissas and the correspondins compressive strengths of the individual 2 by 4-in. cylinders are plotted as the ordinates. The tens ioe briquettes were 1:3 mix and the compression cylinders were 1:2 mix. The straight diagonal ls[1]ng are the "mean regression lines" and the irregular lines are the "raw regression lines." The raw linne quite closely approximate the mean lines. The full lines mark the means of the tension results plotted on the *x*-axis or in the horizontal arrays. The dash lines mark the means of the compression results plotted on the *y*-axis or in the vertical arrays. The marked divergence of the mean regression lines indicate a small degree of correlation, $r = 0.3375 \pm 0.0374$.

ment. They both furnish data that are sufficiently characteristic of the material to be checked by a second specimen with a fairly high degree of accuracy.[2]

[1] J. W. Gowen and H. W. Leavitt, "The Choice of a Mix for Determining the Compressive Strength of Mortars," Maine Technology Experiment Station *Bulletin No. 5*, pp. 1–16, 1923.

[2] Further analysis of these data may be found in Maine Technology Experiment Station *Bulletin No. 7*.

The third question next arises: Does the tension test give the same information about the material which the compression test gives? If so, one or the other of the tests should be discarded as wasted effort. If the tests give different information they both should be conserved. This question may be answered by the correlation method, by constructing a scatter diagram in which the tension results are plotted on one axis and the compression results on the other axis. It is to be understood that each pair of tension and compression tests are made from the same materials. When the correlation coefficient determined from such a diagram is as high as that for the tension of one specimen with the tension of another specimen on the same material (about 0.9) it is proved that the tension and the compression tests show the same properties of the material without giving additional information. If on the other hand the correlation coefficient is much lower, approximately 0.0, then the two tests obviously show different properties of the strength of the mortar. When this question is so studied it is found that the correlation coefficient between tension and compression is much lower than that between tension and tension or between compression and compression. The correlation as actually determined (0.3375 ± 0.0374) shows clearly that both the tension and compression tests develop new information about the strength of a mortar. Fig. 3 illustrates this slight relationship between tension and compression upon 28-day specimens.

The fact that the correlation is 0.337 and not 0.0 shows that the tests are somewhat supplementary; in fact, each has a relatively small underlying common element tending slightly toward a development of like strengths. Thus, the tension and compression tests meet the three requirements and appear to be valuable assets to mortar testing technique.

MORTAR ABRASION TEST

There are undoubtedly other tests which should be added to furnish other independent information concerning the strength of mortars. The wear-resisting qualities of a concrete pavement at once suggest the desirability of determining this characteristic in a mortar test. The authors have in the process of development a method of casting 2-in. spheres of mortar and testing them in abrasion.

Description of Apparatus.—The molds used in preparing the abrasion test specimens (2-in. spheres of 1:2 mortar) are the same as illustrated in Fig. 67, p. 86, *Bulletin No. 35* of the National Research

Council, Vol. 6, Part 4, August, 1923. Three spheres of mortar are cast at one time for each test.

Tentative Method of Test.—The three 2-in. spheres of mortar used in determining the abrasion-resisting qualities of each sand are cured for 24 hours in moist air and 27 days in water. They are then dried for one day at about 100° F. After the three specimens are weighed they are placed in the Deval abrasion cylinder and run for 2000 revolutions at a speed of about 33 r. p. m. They are then reweighed

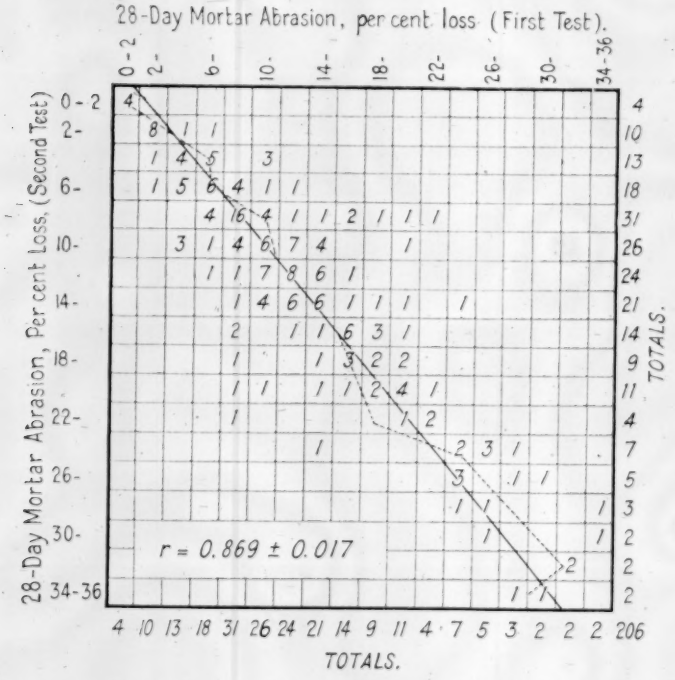

FIG. 4.—Abrasion Tests of Mortar, 28 Days (1:2 Mix, 103 Sands).

This symmetrical correlation table, similar to Figs. 1 and 2, shows the high relationship or ability to check one abrasion test with subsequent tests upon the same materials. The abscissas represent the results of the first abrasion tests and the ordinates, the second abrasion tests. Had a perfect check been obtained all the results would have fallen upon a 45-deg. diagonal line from the upper left-hand corner to the lower right-hand corner of the table and the value of r would have been 1.0 instead of 0.869.

and the loss in weight computed as the percentage of wear. Several periods of time were tried out ranging from one hour to five hours. These data showed the one-hour period to be just as indicative of the wear as the longer periods and it was therefore adopted because of its relative economy of time.

Significance of Mortar Abrasion Test.—This test clearly meets the first requirement, as resistance to wear is a very practical problem

in the study of the value and endurance of the modern cement-concrete pavement. The interrelationship of individual tests when studied by the correlation method shows a high value for the correlation coefficient. Fig. 4 shows this relationship which, as found upon 103 different sands with two abrasion tests upon each, has a correlation coefficient of 0.869 ± 0.017. While the correlation coefficient illustrating the quantitative relationship, or ability to check, for the

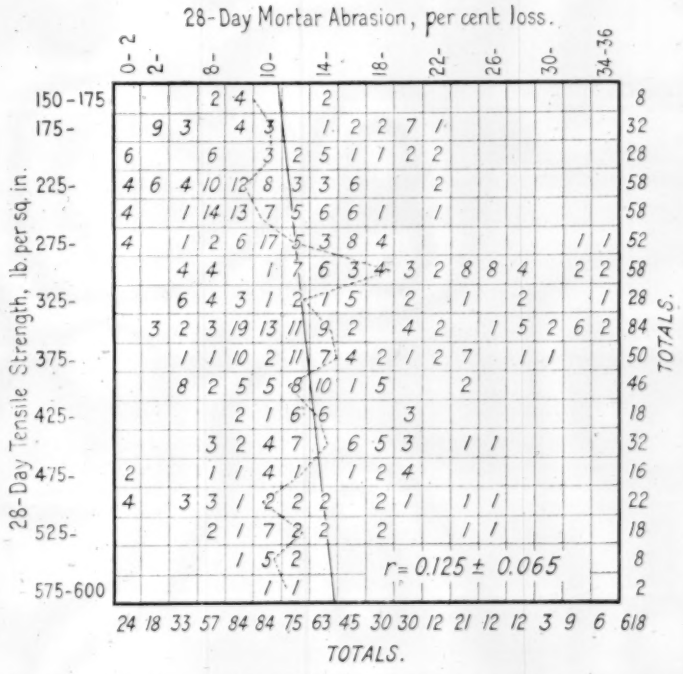

FIG. 5.—"Scatter" Diagram Showing Relationship between Abrasion and Tension Tests of Mortar.

The individual test results in abrasion are plotted as abscissas and the individual test results in tension are plotted as ordinates. Note the wide dispersion or scattering of the results. The correlation coefficient, 0.125 ± 0.065, indicates very little relationship between the two tests.

abrasion test is not as high as that for the tension test (see Fig. 1), it is higher than that for the compression test (see Fig. 2) and is very significant. Thus the second requirement is met. The results obtained in the abrasion test as designed by the authors are true attributes of the mortar.

When this test is correlated with the tension test or with the compression test on the same mortars there is found to exist only a small amount of relationship. Fig. 5 illustrates the relationship of the abrasion and tension tests.

Fig. 6 illustrates the relationship of the abrasion test with the compression test. The mix in each case is 1:2.

The third and final requirement is thus met. The abrasion test therefore furnishes information concerning the strength of the mortar independent of that knowledge furnished by the tension test or the compression test. This new test, then, should add worthwhile information to the sum total of the measures of the strength characteristics of a sand.

CONCLUSIONS

In conclusion the writers wish to state that the description of the methods of analysis herein used, while possibly unfamiliar to

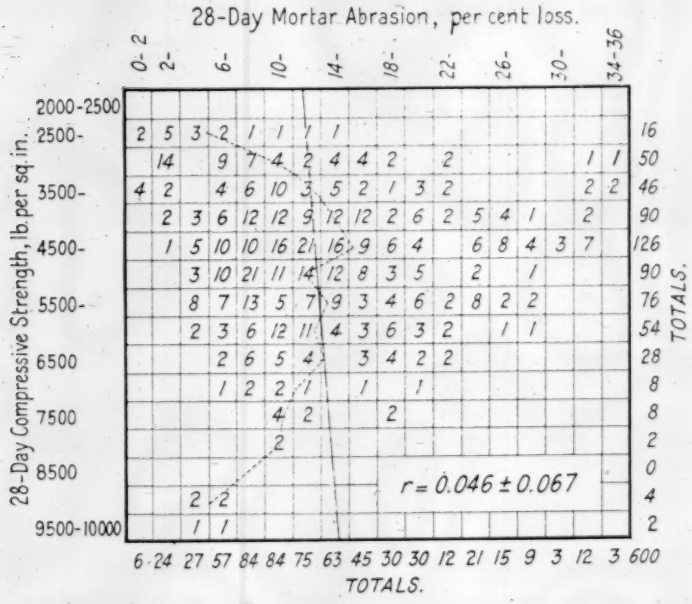

FIG. 6.—"Scatter" Diagram Showing Relationship between Abrasion and Compression Tests of Mortar.

The individual test results in abrasion are plotted as abscissas and the individual test results in compression are plotted as ordinates. The very low relationship here illustrated is measured by the very low value of r, 0.046 ± 0.067.

many engineers at the present time, is not here given because of limited space. The methods are given in such standard texts as referred to above.

The main conclusions drawn from this investigation are:

1. The standard tension test, the standard compression test and

the new abrasion test upon sand mortars give a quantitative measure of important stresses to which a mortar is subjected in many concrete structures.

2. Each of these tests gives concordant results when check tests are performed upon the same materials.

3. Each of these tests gives new information concerning the behavior of the material and there is but very little duplication of fact or information already established by any of the other tests.

4. In the light of the above conclusions it is believed that each of these tests should be performed upon all sands to be used in portland-cement concrete.

DISCUSSION

Mr. Proudley.

MR. C. E. PROUDLEY[1] (*presented in written form by F. H. Jackson*).—In their paper on the significance of the tests of mortars Messrs. Gowen and Leavitt have shown by an unusual method of mathematical analysis the dependence that can be placed in tensile or compressive strength tests of sand and their newly developed sand abrasion test. The criteria for a satisfactory test as given in this paper can be applied safely to any test of mortar, but a question arises as to whether a strength or wear test of the mortar as made in the laboratory is a practical indication of the behavior of the final concrete mixture in so far as it is affected by the fine aggregate.

The real proof of the value of laboratory tests is shown by the extent to which they agree with observations of the behavior of the material in practice. Theoretically, tests may be selected to measure those properties of sands which affect their use in concrete structures.

The authors of the paper under discussion mention three requisites for a satisfactory mortar test. In discussing their first requirement, they state that tension and compression are two of the most important stresses in concrete. We know, however, that the combined influence of the several working stresses caused by loads, temperature and moisture changes can not be predicted with certainty and that our most exhaustive laboratory analyses are therefore not infallible. Instances are on record of the rejection of sands by one or more of the customary mortar tests, whereas tests in concrete proved them quite satisfactory. In a paper on Wear of Concrete Pavements, presented at the 1924 annual meeting,[2] it is shown that there is no relation between the tensile strength ratio of a sand mortar and the compressive strength or modulus of rupture of the concrete. It may be that the only satisfactory way to predetermine the physical properties of concrete resulting from the use of a certain sand will be to make thorough concrete tests on comparatively large sized specimens in the laboratory.

Aside from the grading, there are certain characteristics which are recognized as desirable in sand, chief of which are a surface texture suitable for adhesion of the cement, and hard, tough, durable grains. Were it not for the effect of grading upon the strength of

[1] Junior Assistant Testing Engineer, U. S. Bureau of Public Roads, Washington, D. C.

[2] F. H. Jackson and J. T. Pauls, "Accelerated Wear Tests of Concrete Pavements," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 864 (1924).

mortar, it might be thought that these qualities would be more directly measured by the mortar strength tests, although there would still be the possibility of uncontrollable factors influencing the results to a disastrous extent. Mr. Proudley.

In a recent series of tests, the U. S. Bureau of Public Roads eliminated the effect of size in a number of sands by regrading them to have the same mechanical analysis. Concrete compression tests and mortar strength ratio determinations using the regraded sands gave the following results:

Tensile Strength Ratio on Original Sample	Tensile Strength Ratio on Regraded Sample	Compressive Strength Ratio on Regraded Sample	Compressive Strength on 1:2:4 Concrete Regraded Sand, lb. per sq. in.
124	108	97	2223
113	102	76	2240
120	105	78	2243
134	99	62	2070
132	89	84	2137
125	107	69	2463

There is no relationship apparent in this series between the mortar tests and the compressive strength on the 6 by 12-in. cylinders. In other words, mortar strength ratio except in so far as the result may be due to the effect of grading or the presence of injurious substances is no indication of the strength of the concrete that will be obtained.

The second requirement, which calls for concordant results on duplication of the test, has caused considerable concern among testing laboratories. Although it is not so stated it is supposed that in duplicating their tests the same operator was used for both tests. The results would naturally show close agreement so long as the method of manipulation was kept the same in each case. In elaboration of this second requirement, I should add that it is highly desirable that a test lend itself to such accurate description and be of such simplicity that ability, experience and laboratory facilities be of minimum importance.

Cooperative tests conducted by the Bureau of Public Roads show a wider variation in strength of sand both in tension and compression than is likely to occur in any single laboratory. Using the same sample of sand and cement, 33 laboratories gave an average variation of 9.4 per cent from the average 28-day tensile strength, and fifteen laboratories showed an average variation of 18.3 per cent from the average 28-day compressive strength. A careful operator can check his own work within 2 or 3 per cent. On ordinary sands,

Mr. Proudley.

several operators working in the same laboratory should check within 5 per cent. To be of any commercial value, tests of mortar by several laboratories should agree within 7.5 per cent.

In conclusion, I would suggest that in the absence of more abundant evidence of the unsatisfactory nature of mortar strength tests, and in view of the fact that there is at present no better method of differentiating between good and bad quality of sand other than concrete tests, the practice of making the mortar strength ratio

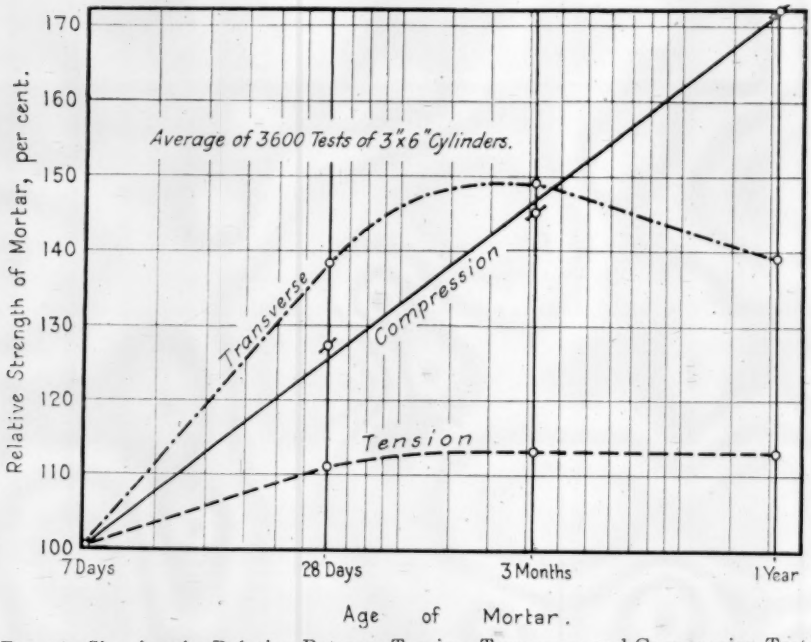

FIG. 1.—Showing the Relation Between Tension, Transverse and Compression Tests of Mortars at Various Ages.

determination should be continued, and the test results given such consideration as they deserve in conjunction with the several other sand tests.

Mr. Hutchinson.

MR. G. W. HUTCHINSON[1] (*presented in written form*).—The paper by Messrs. Gowen and Leavitt calls attention to a matter which is undoubtedly of importance in determining the value of certain tests which will lead to a coordination of the work of the laboratory and that of the field. There is without doubt a place for the several methods of testing now in use, although the relative value of the several tests is more or less a matter of opinion. In preparing this

[1] Eastern Manager, Concrete Department, Celite Products Co., New York City.

discussion, the time available was insufficient to get data into shape on the same basis as outlined in this paper, except to the extent of showing the non-relation in general between the different types of tests in actual use. Mr. Hutchinson.

The accompanying Fig. 1 shows the relation between tension, transverse, and compression tests of mortar mixtures containing

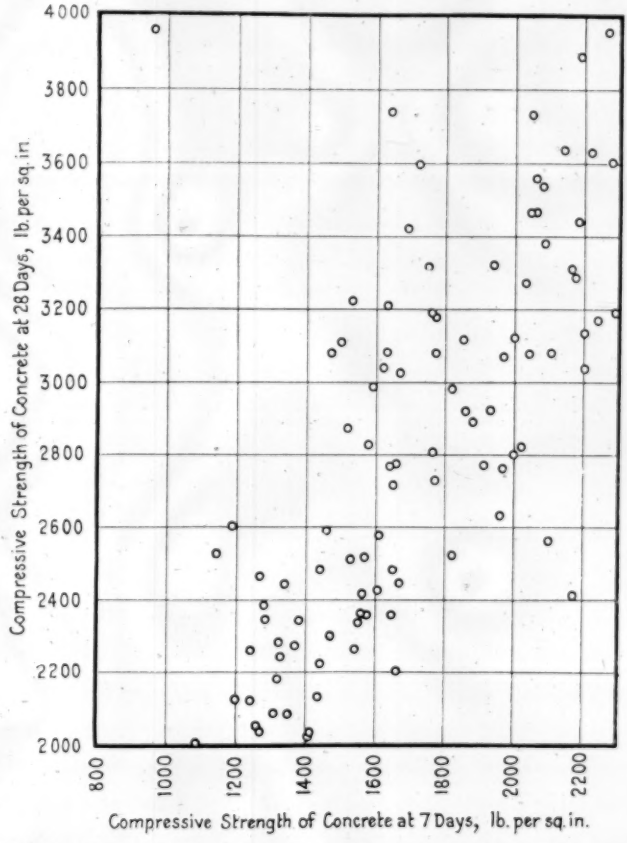

FIG. 2.—Showing Comparison of Compressive Strengths, at 7 and 28 days, of Various Concretes made with Standard Ottowa Sand and Standard Coarse Aggregate.

various amounts of cement and tested at various ages. These curves indicate the average values obtained from 3600 individual tests. The cement content was proportioned by volume of completed mixture, and varies from 20 per cent to neat cement. The results were obtained from the three artificial gradations of fine aggregate usually referred to as fine, medium and coarse. Each point represents the average of 300 specimens. Variation in the direction of these curves

Mr. Hutchinson.

is encountered as the cement content and the aggregates are changed. It will be noted that the general relation between these tests does not change until after the 28-day period.

The application in practice of the results obtained at early ages seems unjustified when reference is made to Fig. 2, and especially so when it is recalled that the compressive strength seems to be the one most consistently affected by age. In this figure are selected a number of typical results taken from the compression tests of standard Ottawa sand and a standard coarse aggregate with various cements. These tests are representative of standard concrete tests by which something over 10,000 samples of natural sands were compared in the process of routine testing. They were all made under standard conditions with control of temperature, consistency and other factors which would tend to affect the actual strength of the mass.

The margin of variation in the increase of relative strength between the 7 and 28-day period is too wide to allow such methods to be considered of much value in governing specifications for field concrete. It is felt that many of the valuable features of concrete are penalized by rigid restriction in the early ages.

We know that different sands show different rates of increase in strength with age and that the same thing is true of cements. We know that different sands act differently with different cements, especially when tested at the early periods. We know that a mortar test of sand at the early periods is not a reliable criterion of its value in concrete. Furthermore, we have evidence that tests of the concrete in place bear practically no relation to the early tests of the individual materials.

These features do not constitute all of the causes for test results being of questionable value as well as for the present agitation for more practical means of determining how to obtain economic concrete in place. I am wondering if our attention is not distracted to a great extent from the main problem before us. In practically all our work we seem to be confining ourselves to average values alone. We accept or reject the individual materials with this in mind. We have specification requirements which can justly be described as extremely rigid in many cases and lax in others. Regardless of this, however, we pass the individual materials on their ability to meet the several standard tests, and in general exert no further control or make no further attempt to tie in the concrete in the laboratory with that secured in the field with the given materials.

The specimens cast at the time the concrete is being placed are also misleading, when compared with the strength and uniformity

of the concrete in place, as determined by cores drilled from the finished structure. An investigation of this nature on thirty different projects discloses the fact that cores drilled from field slabs vary from 65 to over 200 per cent of the strength of cylinders cast from the same batches of concrete from which the cores were taken. This amount of variation is derived from the average of four and five specimens in each case and is not based on single tests. The single tests would vary still more than this. Mr. Hutchinson.

We have been attacking the problem of field concrete in the laboratory for some time. Limited finances have prevented the laboratory from becoming closely associated with the field. It has bred a prejudice against laboratory methods and laboratory control of field concrete. I believe the sooner we realize the extreme variation occurring in field concrete, as indicated by the result of test cores, the sooner we will appreciate that the matter of greatest concern is one of the field rather than one of the laboratory. Laboratory trained forces, however, will be called upon to solve the problem. I am now referring to the abuse of the materials which have passed the desired individual requirements in the laboratory.

Tests of cores drilled from field concrete show that the average variation in strength is from 100 to 125 per cent of the minimum, regardless of the fact that the usual requirements for the individual materials entering into this mass are enforced. The variation has been encountered in cores taken as close as 3 ft. from each other in pavement slabs, and in some cases from the same batch of concrete. The average strength as obtained from all tests of this nature may or may not be satisfactory. When the individual results are considered, the variation from this average cannot be considered economic. The matter should warrant serious consideration and justify means whereby it can be corrected to some degree.

With the result of tests of concrete in place as a basis, the average strength is of minor importance, as a structure seldom fails as a whole. The uniformity of concrete is of more practical importance and exerts considerable influence in determining the economy of the structure. As the uniformity is increased, the average strength requirements may be decreased consistently.

Uniformity of concrete is dependent upon several factors. With acceptable materials from a given source of supply, the two most important factors are accurate proportioning of the materials and the prevention of segregation in the mixed concrete during placement. Any inaccuracy in the proportioning of each size or kind of material changes the ratio of that size or kind to every other in the mass.

Mr. Hutchinson. By drilling test cores from 54 pavement surfaces the results indicated that a considerable part of the variation in field concrete is due to manipulation. In this work four cores were drilled in the cross-section at a given location in each pavement. Two of the cores were drilled within 18 in. of the edges of the pavement and the other two were taken from the middle thirds. The only difference between the concrete from which the two center cores were taken and that on the edges was that the edge concrete required manipulation for placing, whereas that in the middle was merely struck off and finished. The results indicated that the concrete on the edges of the pavement varied more than twice as much in compressive strength as that in the middle. This was true regardless of the fact that the average strengths were the same.

Fortunately, owing to the wider limits of gradation of coarse aggregate in highway work these variations should represent the extreme; however, we do not have the masses of steel to contend with during manipulation such as are generally encountered in other types of general construction. I believe it can be truthfully said that it is not the use but the abuse of concrete that forms the basis for destructive criticism.

It is obvious, that by improving the methods of proportioning cement and aggregate in concrete mixtures, and making it possible to decrease manipulation now necessary in placing, we will tend to improve uniformity, which is the determining factor for securing the most satisfactory concrete.

Mr. Leavitt. MR. H. W. LEAVITT.—I am very glad that the discussion has brought out the points that it did. It shows that in the preparation of this paper we did too good a job. We did not go enough into detail about just what we were explaining. For instance, we used the 28-day test on mortar only. We did not attempt to enter the field of concrete at all. We confined our mortars to a commercial sand mixture. The fact that we used a 28-day test on these sands was because this is the most common test in use at the present time. We, however, have published a bulletin, issued last January,[1] on the relationship between the 7-day and the 28-day test. From the data there presented it is seen that the 28-day test can be predicted very accurately from the 7-day result. Beyond the 28-day period we know nothing so far.

The operators were the same throughout the whole investigation, that is, one man did the mixing of all the sands discussed in this paper.

The question of the grading of different successive batches of sand is, we realize, a rather serious one, and we are at present inves-

[1] *Bulletin No. 10*, Maine Technology Experiment Station.

tigating this to see how the changes in the different sizes of the particles will affect these strengths. Mr. Leavitt.

We do not claim that the abrasion test measures a field condition, but we do claim that it meets a need felt in the State of Maine by providing a means for rejecting certain sands of soft particles, usually flat and friable in character, that may be reduced to a powder in the hand, which neither a tension test nor a compression test as now performed will do.

MESSRS. J. W. GOWEN and H. W. LEAVITT (*author's closure by letter*).—Careful consideration of the written discussion of Messrs. Proudley and Hutchinson lead to several facts of general importance Mr. Proudley raised the question, "Do laboratory tests indicate the behavior of the same materials in the field?" This question is also brought up in our paper; in fact, was raised by us two years ago.[1] It is the basic fundamental assumption behind all testing work. In our judgment, we have to-day no adequate material to approach the solution of this question. It seems probable that when finally properly analyzed, the field results and the laboratory results will be found to be correlated. However, from a research standpoint this lack of data need not trouble us, for the progress of the concrete industry under field conditions has always been dependent upon the thorough understanding of the conditions making for strength or wear as determined in the laboratory. Messrs. Gowan and Leavitt.

Mr. Proudley's discussion of the interrelationship of the tension and compression results is interesting in that it would appear that he missed a fundamental point in our proof, namely, that there is only a small degree of relationship between the tension and compression tests. This fact then leads to the conclusion that the tension and compression tests measure different stresses in mortars.

The series of six tests presented by Mr. Proudley is very interesting in showing what some of the variations might be between tests. It is just this sort of material, when sufficient numbers are collected, which will be valuable in analyzing the problems of the mortar concrete correlations. We suggest that actual strengths, as well as ratios, be always considered in this work.

Mr. Proudley raises the important question of the personal equation of the laboratory. As this question is too complex to be discussed here[2] we will make only one rather categorical statement in support

[1] John W. Gowan and H. Walter Leavitt, "The Choice of a Mix for Determining the Compressive Strength of Mortars," Maine Technology Experiment Station, *Bulletin No. 6*, pp. 1–16, 1924.

[2] See John W. Gowen and H. Walter Leavitt, "The Strength of Mortars in Relation to the Experience and Personal Equation of the Operator," Maine Technology Experiment Station, *Bulletin No. 11*, 1925.

Messrs. Gowan and Leavitt.

and further extension of his statement, namely, that the personal equation enters in every test. Our suggested abrasion test is no more, if as much, subject to the personal equation than the tension and compression tests. Furthermore, it gives more valuable information about the mortar in one of its very important practical stresses.

The concluding paragraph is in entire agreement with our whole argument. We are very much in favor of retaining both the tension and compression tests. In fact, we would go one step further and urge the trial of the abrasion test as one offering possibilities of new information on the practical worth of the materials in the mortar.

The comments of Mr. Hutchinson, while important and instructive, have so little bearing on our paper that it is unfortunate they did not appear as a separate contribution where they might have received more adequate discussion. It is pleasing to us to see that his data on 7-day and 28-day compression tests bear out the conclusion of another paper of ours.[1] It is important to be sure that the results are true random samples of the material. We shall consequently look forward to seeing the results for the full group of 10,000 tests. With the conclusion derived by Mr. Hutchinson that "the application in practice of the results obtained at early ages seems unjustified," we cannot agree. In fact, it appears to us as if there were a good deal of correlation between the 7-day and 28-day compression tests in his data. His Fig. 2 gives a correlation coefficient of 0.655 ± 0.038 or the higher the 7-day strength the higher the average 28-day strength. We doubt further if we can subscribe to his paragraph indicating what we know about mortars and concretes or that cores drilled from pavements would be expected, on the basis of how they were selected, to be comparable. However, discussion of these results should be reserved for another place, for their significance to our paper is but slight.

[1] John W. Gowen, H. Walter Leavitt and Weston S. Evans, "The Prediction of the 28-day Breaking Strengths of Mortars from Their 7-day Results," Maine Technology Experiment Station, *Bulletin No. 10*, 1925.

EFFECT OF SIZE AND SHAPE OF TEST SPECIMEN ON COMPRESSIVE STRENGTH OF CONCRETE

By Harrison F. Gonnerman[1]

Synopsis

This paper describes compression tests to determine the effect of the size and shape of test specimens on the compressive strength of concrete.

The tests were made at 7 days to 1 year on 1755 concrete specimens in a study of the compressive strength of:

1. Cylinders 1½ to 10 in. in diameter and 2 diameters in length, when such factors as size and grading of aggregate, mix, consistency and age were varied over a wide range.
2. Cylinders 12 in. in length ranging from 3 to 10 in. in diameter.
3. Cylinders 6 in. in diameter ranging from 3 to 24 in. in length.
4. Cubes, 6 and 8-in.
5. Prisms, 6 by 12 and 8 by 16-in.

Most of the tests were made at 28 days on 1:5 and 1:3 concrete of relative consistency 1.10, using sand and pebble aggregate graded 0–1½ in. The relative strength of the different forms of specimen was compared with the strength of 6 by 12-in. cylinders from the same concrete.

The tests show that the 6 by 12-in. cylinder recommended by the Society as standard is a satisfactory form of compression specimen for concrete for aggregates up to about 2 in. and that the ratio of diameter of cylinder to maximum size of aggregate should not be less than about 3. The data presented show the relationship between the strengths of the various specimens and the principal conclusions are given at the close of the paper.

Introduction

Standardization of form of test specimen has played an important part in studies of concrete. The 6 by 12-in. cylinder is now generally used for compression tests; however, it is not always practicable to use this form, particularly for specimens cut from structures, therefore it becomes important to know the relative strength of concrete specimens of different sizes and shapes. Considerable data on the effect of size and shape of specimen are available for stone, masonry piers, etc., but comparatively few tests have been carried out on concrete; in the tests reported no attempt was made to differentiate between different types of concrete.

[1] Associate Engineer, Structural Materials Research Laboratory, Lewis Institute, Chicago.

The investigation reported upon in this paper was made to obtain more complete information on the compressive strength of concrete specimens of a wide range of shape and size when such factors as mix, consistency, grading and size of aggregate, and age at test were varied. The tests formed a part of the experimental studies of concrete and concrete materials carried out at the Structural Materials Research Laboratory through the cooperation of the Lewis Institute and the Portland Cement Association.

OUTLINE OF TESTS

This investigation comprised tests on 1755 concrete specimens, in a study of the compressive strength of:

1. Cylinders $1\frac{1}{2}$ to 10 in. in diameter and 2 diameters in length for variations in:

(*a*) Size of aggregate (0 – No. 4 to 0 – 3 in.; Tables I and II).

(*b*) Grading of aggregate (sand and pebbles, sand and limestone, graded 0–$1\frac{1}{2}$ in.; fineness modulus 4.00 to 6.00; Table III).

(*c*) Quantity of cement (1:7 to 1:3 mixes; Table IV).

(*d*) Consistency of concrete (1:5 concrete of relative consistency 0.90 to 1.50; Table V).

(*e*) Age at test (7 days to 1 year; Table VI)

2. Cylinders 12 in. in length for diameters of 3 to 10 in. (Table VII).

3. Cylinders 6 in. in diameter for lengths of 3 to 24 in. (Table VIII).

4. Cubes, 6 and 8-in. (Table IX).

5. Prisms, 6 by 12 in. and 8 by 16 in. (Table IX).

Further details of the tests are given in the tables. Most of the tests were made at age of 28 days on 1:5 and 1:3 concrete of relative consistency 1.10, using sand and pebble aggregate graded 0 – $1\frac{1}{2}$ in.

In general, the maximum size of aggregate in Group No. 1 above was limited to one-half the diameter of the cylinder. A few tests on 2-in. cubes of 1:5 and 1:3 mix made from 0 – No. 4 sand were included in this group.

Aggregate generally consisted of sand and pebbles from Elgin, Ill.; in a few tests, Chicago limestone was used. The sand was well graded from 0 to the No. 4 sieve. The fineness modulus was used as a measure of the size and grading of the aggregate. This function is the sum of the percentages in the sieve analysis divided by 100, when the sieve analysis is expressed as percentages coarser than each of the following sieves: No. 100, 48, 28, 14, 8, 4, $\frac{3}{8}$-in., $\frac{3}{4}$-in., $1\frac{1}{2}$-in., 3-in., etc. Each sieve has a clear opening double that of the next smaller

sieve. The coarse aggregates were screened into five sizes and recombined to pre-determined sieve analyses.

Concrete was mixed by hand on a metal plate in batches of about 1 cu. ft. The proportions are by volume; one volume of cement to a given number of volumes of dry and rodded aggregate mixed as used. The quantities for each batch were determined by weight; 94 lb. of cement were considered to be 1 cu. ft. Water from Lake Michigan was used in mixing.

Consistency of the concrete was measured by means of the flow-table, using fifteen $\frac{1}{2}$-in. drops in 10 seconds. The dimensions of the truncated-cone metal form used in the flow tests were: top diameter $6\frac{3}{4}$ in., bottom diameter 10 in., height 5 in. The base diameter of the mass of concrete after the test expressed as a percentage of the original diameter is the flow. The flow for relative consistency 1.00 was 170 to 190. Concrete of relative consistency 0.90 contained 10 per cent less, and of relative consistency 1.10 contained 10 per cent more mixing water than concrete of relative consistency 1.00; similarly for other consistencies. Most of the tests were made on concrete of relative consistency 1.10 (flow about 220).

Test Specimens.—The dimensions of the cylinders, prisms and cubes are given in the tables. The test specimens were molded in metal or metal-lined forms and so far as possible in the same manner. For specimens of length equal to or greater than 2 diameters the concrete was placed in the forms in three layers; shorter specimens in two layers. Each layer was rodded 30 times with a round bullet-pointed steel rod. A $\frac{1}{4}$-in. rod was used on specimens 3 in. or less in diameter, a $\frac{5}{8}$-in. rod on specimens 4 to 6 in. and a 1-in. rod on specimens larger than 6 in. The forms rested on plane cast-iron plates. Three to four hours after molding, the specimens were capped with neat cement paste which had been allowed to stand 3 or 4 hours. The caps were made plane by means of machined cast-iron plates.

The forms were removed after 16 to 20 hours and the specimens then cured in a moist room until tested. The specimens were tested in a damp condition in testing machines of 40,000, 200,000 or 300,000-lb. capacity, depending on the size of specimen. The load was applied through a spherical bearing block placed on top of the specimen.

DISCUSSION OF TESTS

Basis of Comparison.—The strengths of all specimens were compared with those of the 6 by 12-in. cylinders by means of strength ratios, that is, the ratio of the strength, expressed as a percentage, of a given form of specimen to that of the 6 by 12-in. cylinder from

TABLE I.—EFFECT OF SIZE OF CYLINDER FOR DIFFERENT SIZES OF AGGREGATE.

Aggregate: sand and pebbles from Elgin, Ill. Fineness moduli of mixed aggregate: 0 - No. 4, 3.00; 0 - $\frac{3}{8}$ in. 4.00; 0 - $\frac{3}{4}$ in., 5.00; 0 - $1\frac{1}{2}$ in., 5.50.

Cement: a mixture of four brands purchased in Chicago.

Relative consistency 1.10. Consistency measured by means of flow table. Sufficient water was used to give a flow of 170 to 190 per cent for a relative consistency of 1.00. Concrete of 1.10 consistency contained 10 per cent more water, and gave a flow of about 220.

Concrete mixed by hand with shovels on metal plate in batches of about 1 cu. ft.

Specimens cured in moist room until day of test; tested damp.

Strength-ratio is the ratio, expressed as a percentage, of the strength of a given size of cylinder to that of the 6 by 12-in. cylinder from the same concrete.

Age at test, 28 days.

Each value is the average of 5 to 20 tests made on different days unless otherwise noted.

See Fig. 1.

Size of Aggregate	Water-Ratio	Compressive Strength, lb. per sq. in.									Strength-Ratio, per cent								
		2-in. Cube	$1\frac{1}{2}$ by 3 in.	2 by 4 in.	3 by 6 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.	2-in. Cube	$1\frac{1}{2}$ by 3 in.	2 by 4 in.	3 by 6 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
MIX 1:5 BY VOLUME																			
0 - No. 4.	1.36	1220	1430	1310	1510	1420	1240	1200	1190	1250	102	119	109	126	118	103	100	99	104
0 - $\frac{3}{8}$ in.	1.15	.....	.....	1740	1960	2110	2150	2120	1980	1840	...	...	82	92	100	101	100	93	87
0 - $\frac{3}{4}$ "	0.99	.....	.....	.....	2730	2660	2700	2810	2700	2560	...	...	...	97	95	96	100	96	91
0 - $1\frac{1}{2}$ "	0.93	.....	.....	.....	.....	2950	3080	3020[a]	2880	2700	...	...	...	...	98	102	100	95	88
MIX 1:3 BY VOLUME																			
0 - No. 4.	0.87	2920	2730	2690	3280	2930	3000	2960	2700	2810	99	92	91	111	99	101	100	91	95
0 - $\frac{3}{8}$ in.	0.75	.....	.....	3510	3890	3760	3920	3870	3630	3570	...	...	91	100	97	101	100	94	92
0 - $\frac{3}{4}$ "	0.67	.....	.....	.....	4480	4840	4650	4680	4360	4230	...	...	...	96	103	99	100	93	90
0 - $1\frac{1}{2}$ "	0.65	.....	.....	.....	.....	4860	4800	4520[b]	4450	4010	...	...	...	...	108	106	100	99	89

[a] Average of 30 tests; same value occurs in all tables.

[b] Average of 25 tests; same value occurs in Tables IV, VII and VIII.

TABLE II.—EFFECT OF SIZE OF CYLINDER FOR DIFFERENT AGGREGATES.

Mix: 1:5 by volume.

Aggregate: Fine, Elgin sand; Coarse, Elgin pebbles and Chicago limestone.

Relative consistency, 1.10.

Age at test, 28 days.

Each value is the average of 10 tests made on different days, unless otherwise noted.

For further details, see notes accompanying Table I.

See Fig. 1.

Aggregate			Water-Ratio	Flow, per cent	Compressive Strength, lb. per sq. in.						Strength-Ratio, per cent					
Kind	Size, in.	Fineness Modulus			3 by 6 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.	3 by 6 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
Sand and Pebbles	0 - $\frac{3}{4}$	5.00	0.99	218	2730	2660	2700	2810	2700	2560	97	95	96	100	96	91
	0 - $1\frac{1}{2}$	5.50	0.93	221	.....	2950	3080	3020[a]	2880	2700	..	98	102	100	95	89
	0 - 3	6.00	0.93	221	.....	.....	.....	2830	2620	2640	..	...	...	100	93	93
Sand and Limestone	0 - $\frac{3}{4}$	5.00	1.05	220	2140	2320	2300	2240	2110	2230	96	104	103	100	94	100
	0 - $1\frac{1}{2}$	5.50	0.97	217	.....	2640	2440	2530	2610	2560	..	104	96	100	103	101
	0 - 3	6.00	0.94	217	.....	.....	.....	2430	2280	2520	..	....	....	100	94	104

[a] Average of 30 tests; same value occurs in all tables.

TABLE III.—EFFECT OF SIZE OF CYLINDER FOR DIFFERENT GRADINGS OF AGGREGATE.

Mix: 1:5 by volume.
Relative consistency, 1.10.
Age at test, 28 days.
Each value is the average of 5 tests made on different days unless otherwise noted.
For further details see notes accompanying Table I.
See Fig. 2.

Aggregate			Water-Ratio	Flow, per cent	Compressive Strength, lb. per sq. in.					Strength-Ratio, per cent				
Kind	Size, in.	Fineness Modulus			4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
Elgin Sand and Pebbles	0 – 1½	4.00	1.24	218	1810	1830	1580	1490	1590	115	116	100	94	101
		4.50	1.15	220	2270	2170	2000	1960	1870	114	109	100	98	94
		5.00	1.05	216	2740	2780	2630	2760	2140	104	106	100	105	81
		5.50	0.93	221	2950	3080	3020[a]	2880	2700	98	102	100	95	89
		5.75	0.87	218	3460	3500	3750	3330	3020	92	93	100	89	81
		6.00	0.87	216	3370	3110	3420	2960	3010	99	91	100	87	88
Elgin Sand and Chicago Limestone	0 – 1½	4.00	1.24	216	1730	1750	1670	1720	1450	104	105	100	103	87
		4.50	1.16	218	2140	2150	2130	2020	1820	100	101	100	95	85
		5.00	1.05	216	2470	2510	2650	2440	2390	93	95	100	92	90
		5.50	0.97	217	2640	2440	2530	2610	2560	104	96	100	103	101
		5.75	0.94	216	2820	2410	2570	2480	2580	110	94	100	97	100
		6.00	0.91	207	2600	2290	2460	2310	2490	106	93	100	94	101

[a] Average of 30 tests: same value occurs in all tables.

TABLE IV.—EFFECT OF SIZE OF CYLINDER FOR DIFFERENT MIXES.

Aggregate: Elgin sand and pebbles graded 0 – 1½ in. (F. M. = 5.50).
Relative consistency, 1.10.
Age at test, 28 days.
Each value is the average of 5 tests made on different days unless otherwise noted.
For further details see notes accompanying Table I.
See Fig. 2.

Mix by Volume	Water-Ratio	Flow, per cent	Compressive Strength, lb. per sq. in.					Strength-Ratio, per cent				
			4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
1:7.....	1.24	218	2140	2070	2160	1790	1600	99	96	100	83	74
1:5.....	0.93	221	2950	3080	3020[a]	2880	2700	98	102	100	95	89
1:4.....	0.78	223	4460	4330	4150	3770	3530	107	104	100	91	85
1:3.....	0.65	217	4860	4800	4520[b]	4450	4010	108	106	100	98	89

[a] Average of 30 tests; same value occurs in all Tables.
[b] Average of 25 tests; same value occurs in Tables I, VII and VIII.

TABLE V.—EFFECT OF SIZE OF CYLINDER FOR DIFFERENT CONSISTENCIES.

Mix 1:5 by volume.
Aggregate: Elgin sand and pebbles graded 0 – 1½ in. (F. M. = 5.50).
Age at test, 28 days.
Each value is the average of 5 tests made on different days unless otherwise noted.
For further details see notes accompanying Table I.
See Fig. 2.

Relative Consistency	Water-Ratio	Flow, per cent	Compressive Strength, lb. per sq. in.					Strength-Ratio, per cent				
			4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
0.90.....	0.79	136	3110	3330	3310	3310	2950	94	101	100	100	89
1.00.....	0.88	180	3120	3240	3060	3110	3020	102	106	100	102	99
1.10.....	0.93	221	2950	3080	3020[a]	2880	2700	98	102	100	95	89
1.25.....	1.09	...	2080	2160	2250	2500	2090	92	96	100	111	93
1.50.....	1.31	...	1590	1580	1710	1630	1560	93	92	100	95	91

[a] Average of 30 tests; same value occurs in all tables.

the same concrete and tested at the same age. The 28-day strengths of 6 by 12-in. cylinders of 1:5 and 1:3 mixes of relative consistency 1.10 and 0 – 1½-in. sand and pebble aggregate, based on three sets of specimens made on different days (at intervals of 4 to 9 days) during the period of molding were:

SET	COMPRESSIVE STRENGTH OF CONCRETE, LB. PER SQ. IN. 1:5 MIX	1:3 MIX
1	3070 (10 tests)	4980 (5 tests)
2	2990 " "	4210 (10 ")
3	3000 " "	4600 " "
Weighted Average	3020 (30 tests)	4520 (25 tests)

For other conditions, the strengths of 6 by 12-in. cylinders are the average of five or ten tests.

Effect of Size of Cylinders Two Diameters in Length.—Data on compressive strength of cylinders two diameters in length for different sizes, gradings and kinds of aggregate, mixes, consistencies and ages are given in Tables I to VI and in Figs. 1 and 2. In these tests the diameters ranged from 1½ to 10 in.

In general, lower strengths were obtained from the larger cylinders. The decrease in strength was not important for diameters up to 6 in., but was more pronounced for larger diameters, as shown in the following table of strength-ratios:

Size of Cylinder	1½ by 3 in.	2 by 4 in.	3 by 6 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
Average Strength-Ratio, per cent of strength of 6 by 12-in. cylinder	105	93	103	101	101	100	96	92
Number of Tests	39	78	77	176	182	248	204	184

These strength-ratios are the average for all conditions of test, except that values for a few mixtures which were not workable and for a few large cylinders whose strengths were beyond the capacity of the testing machine are omitted. These values are plotted as the dash-line curve in each group in Figs. 1 and 2. The solid-line curve is based on the average strength-ratios for the particular group, and in many cases coincides with the grand average curve. For aggregate graded 0 – 1½ in. or finer, almost identical results were obtained with 4 by 8, 5 by 10 and 6 by 12-in. cylinders. The 8 by 16-in. and 10 by 20-in. cylinders gave strengths 4 and 8 per cent lower than the 6 by 12-in.

Satisfactory strengths were obtained with aggregate graded up to 1½ in. in cylinders having diameters as low as 4 in., and with 3-in.

aggregate in cylinders having diameters as low as 6 in. However, in cases where the diameter of the cylinder was less than three to four times the maximum size of aggregate, it was difficult to place the

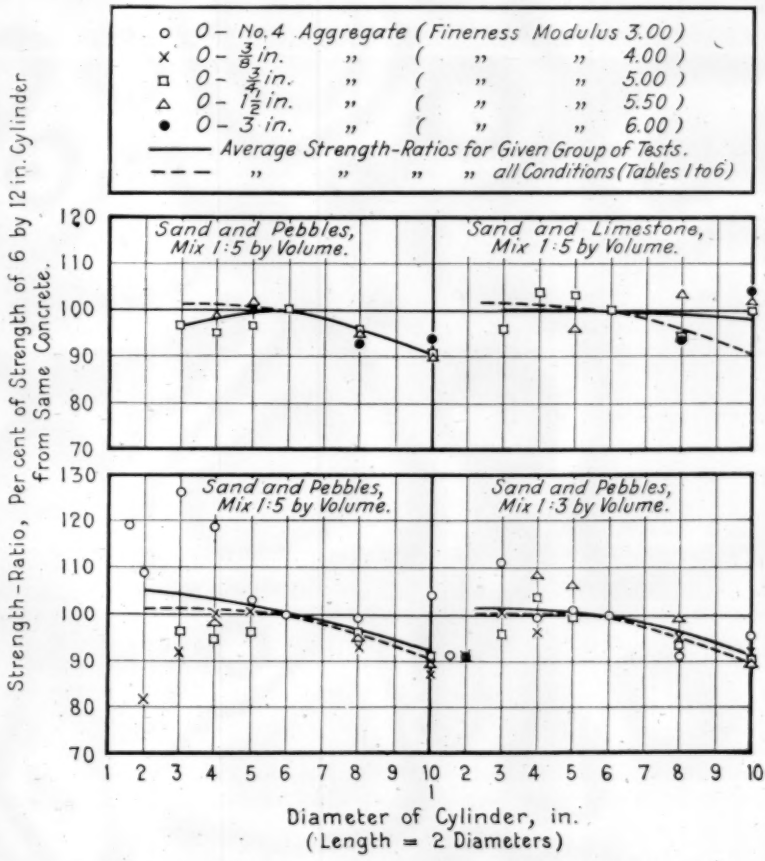
O – No. 4 Aggregate (Fineness Modulus 3.00)
O – 3/8 in. " (" " 4.00)
O – 3/4 in. " (" " 5.00)
O – 1 1/2 in. " (" " 5.50)
O – 3 in. " (" " 6.00)
Average Strength-Ratios for Given Group of Tests.
" " " " all Conditions (Tables I to 6)
Sand and Pebbles, Mix 1:5 by Volume.
Sand and Limestone, Mix 1:5 by Volume.
Sand and Pebbles, Mix 1:5 by Volume.
Sand and Pebbles, Mix 1:3 by Volume.
Strength-Ratio, Per cent of Strength of 6 by 12 in. Cylinder from Same Concrete.
Diameter of Cylinder, in. (Length = 2 Diameters)

FIG. 1.—Effect of Size of Cylinders 2 Diameters in Length for Different Sizes of Aggregate.

Compression tests of concrete cylinders.
Relative consistency 1.10; age at test 28 days.
Each value is the average of 5 to 30 tests.
Data from Tables I and II. Compare Fig. 2.

concrete in a homogeneous mass and it is recommended that in tests of concrete a ratio of diameter of cylinder to maximum size of aggregate of at least three be maintained in order to secure homogeneous specimens.

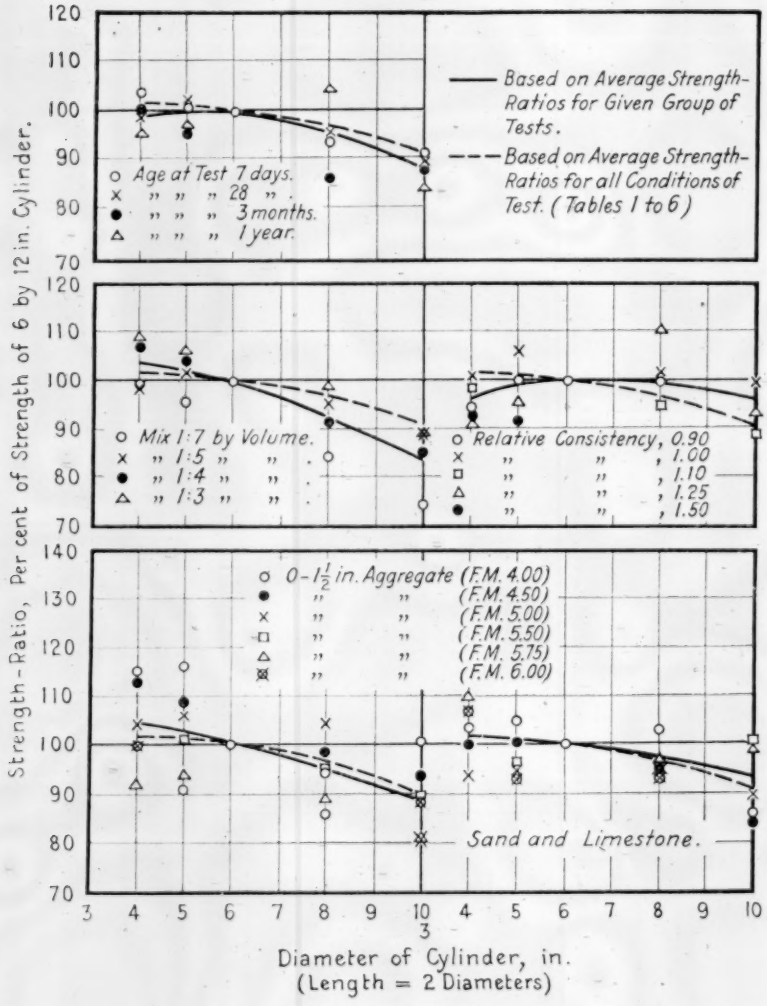
Based on Average Strength-Ratios for Given Group of Tests.
Based on Average Strength-Ratios for all Conditions of Test. (Tables 1 to 6)
Age at Test 7 days.
" " " 28 " .
" " " 3 months.
" " " 1 year.
Mix 1:7 by Volume.
" 1:5 " " .
" 1:4 " " .
" 1:3 " " .
Relative Consistency, 0.90
" " , 1.00
" " , 1.10
" " , 1.25
" " , 1.50
0-1½ in. Aggregate (F.M. 4.00)
" " (F.M. 4.50)
" " (F.M. 5.00)
" " (F.M. 5.50)
" " (F.M. 5.75)
" " (F.M. 6.00)
Sand and Limestone.
Strength-Ratio, Per cent of Strength of 6 by 12 in. Cylinder.
Diameter of Cylinder, in.
(Length = 2 Diameters)

FIG. 2.—Effect of Size of Cylinders 2 Diameters in Length.

Compression tests of concrete cylinders; plotted values for 10 by 20-in. cylinders at 3 months and 1 year are low as strengths were above capacity of machine.

The principal variable is indicated for each group; unless otherwise noted the concrete was 1: 5 mix, relative consistency 1.10, sand and pebble aggregate graded 0 – 1½ in. (fineness modulus 5.50).

Each value is the average of 5 to 30 tests; age at test 28 days unless otherwise noted.

Data from Tables II to VI. Compare Fig. 1.

TABLE VI.—EFFECT OF SIZE OF CYLINDER AT DIFFERENT AGES.

Mix: 1:5 by volume.
Aggregate: Elgin sand and pebbles graded 0 – 1½ in. (F. M. = 5.50).
Relative consistency 1.10 (water-ratio 0.93, flow about 220 per cent).
Each value is the average of 10 tests unless otherwise noted.
For further details see notes accompanying Table I.
See Figs. 2 and 4.

Age at Test	Compressive Strength, lb. per sq. in.					Strength-Ratio, per cent				
	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.	4 by 8 in.	5 by 10 in.	6 by 12 in.	8 by 16 in.	10 by 20 in.
7 days	1600	1550	1540	1450	1410	104	101	100	94	92
28 days	2950	3080	3020[a]	2880	2700	98	102	100	95	89
3 months	4480	4260	4480	3870	3950[b]	100	95	100	86	88[b]
1 year	4860	5000	5130	5390	4280[c]	96	97	100	105	84[c]

[a] Average of 30 tests; same value occurs in all tables.
[b] Value low; four cylinders loaded to capacity of testing machine without failure.
[c] Cylinders loaded to capacity of testing machine without failure; values for compressive strength and strength-ratio are calculated from load at capacity of machine.

TABLE VII.—EFFECT OF DIAMETER FOR CYLINDERS 12 IN. IN LENGTH.

Aggregate: Elgin sand and pebbles graded 0 – 1½ in. (F. M. = 5.50).
Relative consistency, 1.10.
Age at test, 28 days.
Each value is the average of 10 tests made on different days unless otherwise noted.
For further details see notes accompanying Table I.
See Fig. 3.

Mix by Volume	Water-Ratio	Flow, per cent	Compressive Strength, lb. per sq. in.						Strength-Ratio, per cent					
			3 by 12 in.	4 by 12 in.	5 by 12 in.	6 by 12 in.	8 by 12 in.	10 by 12 in.	3 by 12 in.	4 by 12 in.	5 by 12 in.	6 by 12 in.	8 by 12 in.	10 by 12 in.
1:5	0.93	221	2640	2760	2960	3020[a]	3010	2800	87	91	98	100	100	93
1:3	0.65	217	4210	4540	4520	4520[b]	4490	4280[c]	93	100	100	100	99	95[c]

[a] Average of 30 tests; same value occurs in all tables.
[b] Average of 25 tests; same value occurs in Tables I, IV and VIII.
[c] Cylinders loaded to capacity of testing machine without failure; values for compressive strength and strength-ratio are calculated from load at capacity of machine.

TABLE VIII.—EFFECT OF LENGTH OF 6-IN. CYLINDERS.

Aggregate: Elgin sand and pebbles graded 0 – 1½ in. (F. M. = 5.50).
Relative consistency, 1.10.
Age at test, 28 days.
Each value is the average of 10 tests made on different days unless otherwise noted.
For further details see notes accompanying Table I.
See Fig. 3.

Mix by Volume	Water-Ratio	Flow, per cent	Compressive Strength, lb. per sq. in.							Strength-Ratio, per cent						
			6 by 3 in.	6 by 6 in.	6 by 9 in.	6 by 12 in.	6 by 15 in.	6 by 18 in.	6 by 24 in.	6 by 3 in.	6 by 6 in.	6 by 9 in.	6 by 12 in.	6 by 15 in.	6 by 18 in.	6 by 24 in.
1:5	0.93	221	5970	3380	3010	3020[a]	2970	2900	2770	198	112	100	100	98	96	92
1:3	0.65	217	7160	....	4350	4520[b]	3980	4040	4030	168	...	96	100	88	89	89

[a] Average of 30 tests; same value occurs in all tables.
[b] Average of 25 tests; same value occurs in Tables I, IV and VII.

Effect of Ratio of Length to Diameter of Cylinder.—Table VII gives data of tests on cylinders 12 in. in length ranging from 3 to 10 in. in diameter, and Table VIII gives data of tests on cylinders 6 in. in diameter ranging from 3 to 24 in. in length. Two concrete mixes, 1:5 and 1:3, each of relative consistency 1.10 were used. The aggregate was sand and pebbles graded 0 – 1½ in. The specimens were tested at 28 days.

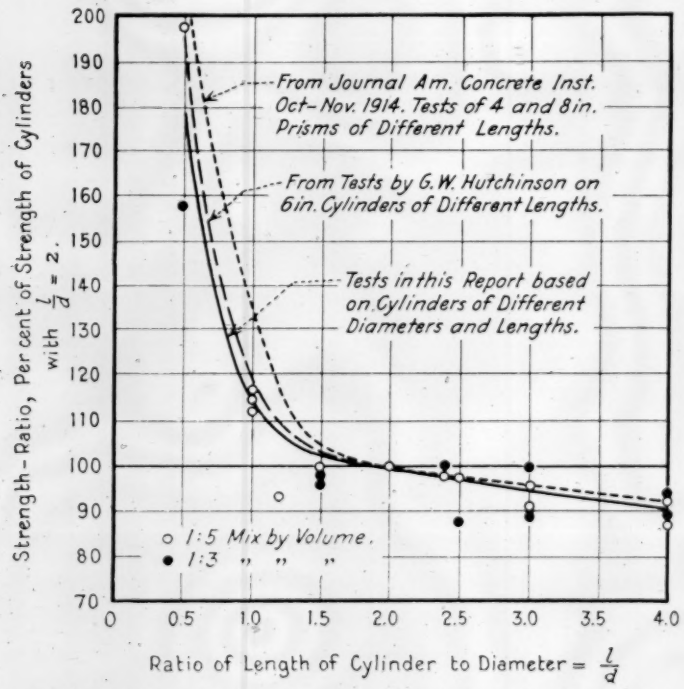

FIG. 3.—Relation of Length and Diameter of Specimen to Compressive Strength.

Compression tests of concrete cylinders and cubes from Tables VII, VIII and IX.
Strength-ratios for tests in this report based on 6 by 12-in. cylinders.
Relative consistency 1.10.
Age at test, 28 days.

The results of these tests are in agreement when the ratio of length to diameter is taken into account. The relation between the ratio of length to diameter of cylinder and the strength of 6 by 12-in. cylinders is shown by the solid-line curve in Fig. 3. For ratios of length to diameter, l/d, of 1.5 to 2.5, the strengths were generally within 5 per cent of that of the 6 by 12-in. cylinder, $l/d = 2$. For l/d less than about 1.25 there was a marked increase in strength; for

a ratio of 0.5, the strength was 178 per cent of that of the 6 by 12-in. cylinder used as the basis of comparison. For ratios of 3.0 and 4.0 the strengths were 95 and 90 per cent of that for the 6 by 12-in. cylinder. The difference in strengths of cylinders having ratios of length to diameter between 1.5 and 2.5 was not important.

For the purpose of comparison, curves based on tests of a similar nature from other sources are also shown in Fig. 3. The dotted curve

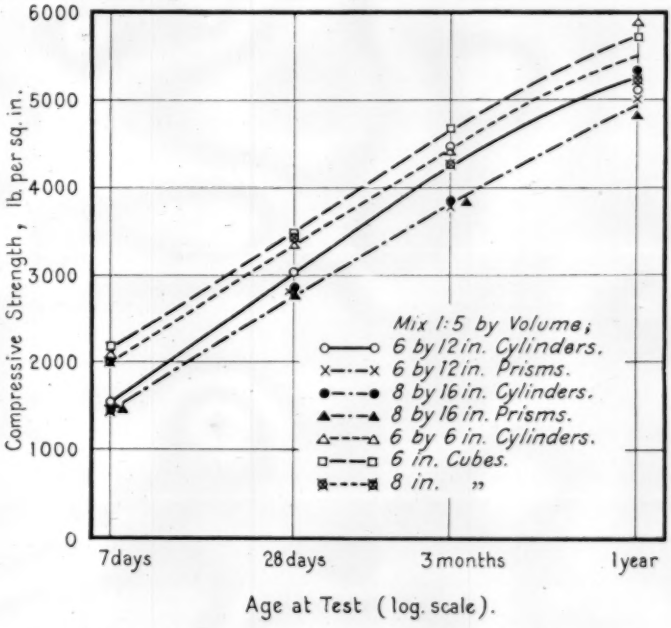

FIG. 4.—Effect of Age on Compressive Strength of Concrete.

Compression tests of concrete cylinders, cubes and prisms.
Relative consistency 1.10.
Each value is the average of 10 to 30 tests.
Data from Tables VI and IX.

is reproduced from the Journal of the American Concrete Institute, October-November, 1914, and is a composite curve based on compression tests of 4 by 4-in. and 8 by 8-in. concrete prisms of different lengths carried out at the Massachusetts Institute of Technology and at the Universities of Illinois and Wisconsin. The prisms were of approximately 1:2:4 concrete of relative consistency 1.00 and were tested at an age of 3 months. The dash-line curve is based on tests

by G. W. Hutchinson[1] on 6-in. cylinders ranging in length from 3 to 12 in., from concretes of different qualities and tested at ages of 7 days to 2 months. The tests by Hutchinson are in close agreement with those in this report. The curve for the American Concrete Institute tests is higher than the other two, but the discrepancies are not great for ratios of length to diameter greater than about 1.25.

Comparison of Cylinders, Cubes and Prisms.—Table IX and Fig. 4 give data of tests on 6 and 8-in. cylinders, prisms and cubes. The concrete was a 1:5 mix of relative consistency 1.10; the aggregate sand and pebbles, graded 0 – 1½ in.

The compressive strengths of the 6 by 6-in. cylinders averaged 15 per cent and of the 6 and 8-in. cubes 18 per cent and 13 per cent

TABLE IX.—COMPARISON OF CYLINDERS, CUBES AND PRISMS.

Mix 1:5 by volume.
Aggregate: Elgin sand and pebbles graded 0 – 1½ in. (F. M. = 5.50).
Relative consistency, 1.10 (water-ratio 0.93, flow about 220 per cent).
Each value is the average of 10 tests made on different days unless otherwise noted.
For further details see notes accompanying Table I.
See Fig. 4.

Age at Test	Compressive Strength, lb. per sq. in.							Strength-Ratio, per cent						
	6 by 6-in. Cyl.	6-in. Cube	8-in. Cube	6 by 12 in.		8 by 16 in.		6 by 6-in. Cyl.	6-in. Cube	8-in. Cube	6 by 12 in.		8 by 16 in.	
				Cyl.	Prism	Cyl.	Prism				Cyl.	Prism	Cyl.	Prism
7 days...	2030	2160	2000	1540	1460	1450	1450	132	140	130	100	95	94	94
28 " ...	3380	3490	3470	3020[a]	2820	2880	2760	112	116	115	100	93	95	91
3 months..	4420	4680	4290	4480	3820	3870	3820	99	105	96	100	85	86	85
1 year.....	5900	5740	5250[b]	5130	5080	5390	4830[c]	115	112	102[b]	100	99	105	94[c]

[a] Average of 30 tests; same value occurs in all tables.
[b] Cubes loaded to capacity of testing machine without failure; values for compressive strength and strength-ratio are calculated from load at capacity of machine.
[c] Value low; three prisms loaded to capacity of testing machine without failure.

higher than those of the 6 by 12-in. cylinders. The strength-ratios for the 8 by 16-in. cylinders in this group of tests ranged from 86 per cent at 3 months to 105 per cent at 1 year and averaged 95 per cent, as compared with 96 per cent given above for all conditions of test. The strengths for the 6 by 12-in. and 8 by 16-in. prisms were lower at all ages than those of the 6 by 12-in. cylinders, averaging 7 per cent lower for 6 by 12-in. prisms and 9 per cent for the 8 by 16-in. prisms. The lower strengths for the prisms may be attributed to the lack of uniform distribution of stress over the section due to the unrestrained corners of the specimens.

In the American Concrete Institute tests mentioned above, the strength of cubes averaged 37 per cent greater than that of prisms of

[1] See "Correction Data for Comparative Test Results from Field Specimens," *Proceedings*, Am. Concrete Inst., p. 191 (1923).

height equal to two times the width. If the cube strengths reported in this paper are compared directly with the prism strengths they average 127 and 126 per cent, respectively, of the strength of corresponding prisms. This difference in method of comparison probably accounts for the discrepancies in Fig. 3 between the American Concrete Institute tests and those by Hutchinson and by our Laboratory. A few tests on 2-in. cubes are given in Table I for 1:5 and 1:3 mixes using 0 – No. 4 sand; the strengths of the cubes were about the same as those of 6 by 12-in. cylinders.

Effect of Age of Concrete.—Data of effect of age on compressive strength of a 1:5 mix of relative consistency 1.10 (water-ratio 0.93) are given in Table VI and IX. The compressive strength of concrete cured in a moist room until tested increased with age for all forms of specimen (see Fig. 4). The average compressive strengths for the five sizes of cylinders 2 diameters in length in Table VI, and for the 6 by 12 and 8 by 16-in. prisms and the 6 and 8-in. cubes in Table IX were as follows:

Age	Cylinders		Prisms		Cubes	
	Compressive Strength, lb. per sq. in.	Percentage of 28-day Strength	Compressive Strength, lb. per sq. in.	Percentage of 28-day Strength	Compressive Strength, lb. per sq. in.	Percentage of 28-day Strength
7 days	1510	52	1460	52	2080	60
28 days	2930	100	2790	100	3480	100
3 months	4270	146	3820	137	4480	129
1 year	5100	174	5080	182	5740	165

For the cylinders and prisms the percentages of the 28-day strength were in close agreement; the 7-day strength averaged 52 per cent, the 3-month 142 per cent and the 1-year 178 per cent of the 28-day strength. The percentages for the cubes are somewhat higher than for the cylinders and prisms at 7 days and lower at 3 months and 1 year.

Uniformity of Results.—The uniformity of the test results was judged by the mean variation, the average variation of the strengths of the individual specimens from the average strength of the set. A study of the mean variations showed that for cylinders 5 to 10 in. in diameter and 2 diameters in length the uniformity in results was approximately the same and was in general somewhat better than for the smaller sizes. Mixtures which were not workable generally gave erratic results, especially in cylinders 4 in. and less in diameter. Cubes and prisms gave about the same degree of uniformity as cylinders.

CONCLUSIONS

The principal conclusions from the tests are:

1. The 6 by 12-in. cylinder generally used for compression tests of concrete, as recommended in the Standard Methods of Making and Storing Specimens of Concrete in the Field (Serial Designation: C 31 – 21) and in the Tentative Methods of Making Compression Tests of Concrete (Serial Designation: C 39 – 21 T) of the American Society for Testing Materials, is a satisfactory form of specimen. However, because of the likelihood of non-uniform placing it is recommended that the use of this size of cylinder be limited to aggregates 2 in. or less in diameter.

2. The 4 by 8-in. or 5 by 10-in. cylinders are suitable for the smaller sizes of aggregate. The ratio of diameter of cylinder to maximum size of aggregate should not be less than about 3. For aggregates larger than 2 in., 8 by 16-in. cylinders or larger should be used.

3. For cylinders of length equal to 2 diameters, lower strengths were generally obtained with the larger cylinders. The decrease in strength with size of cylinder was not important for diameters of 6 in. or less; 8 by 16-in. and 10 by 20-in. cylinders gave 96 and 92 per cent of the strength of 6 by 12-in. cylinders.

4. Concrete cylinders having a ratio of length to diameter of from 0.5 to 4.0 gave the following average strength-ratios at 28 days:

Ratio of Length to Diameter	0.5	1.0	1.25	1.5	2.0	3.0	4.0
Strength-Ratio, percentage of strength of 6 by 12-in. cylinder	178	115	107	103	100	95	90

In general, these strength-ratios agree with those reported by other investigators. The difference in strengths of cylinders having ratios of length to diameter between 1.5 and 2.5 was not important.

5. The 6 and 8-in. cubes tested at ages of 7 days to 1 year gave strengths averaging 18 and 13 per cent higher than 6 by 12-in. cylinders. The strengths for 6 by 12 and 8 by 16-in. prisms were lower at all ages than that for 6 by 12-in. cylinders; the strength-ratios averaged 93 and 91 per cent, respectively.

6. For all forms of specimens the compressive strength increased with age for moist curing. For cylinders and prisms of length equal to twice the diameter or width, the 7-day, 3-month and 1-year strengths averaged 52, 142 and 178 per cent of the 28-day strength for 1:5 concrete. The corresponding percentages for 6 and 8-in. cubes were 60, 129 and 165 per cent.

DISCUSSION

MR. E. E. BUTTERFIELD[1] (*presented in written form*).—Mr. Gonnerman describes the strength relations for various sizes and shapes of test specimens molded in the laboratory. We have reasons to believe from the work of Mattimore in Pennsylvania as well as from our work in the Borough of Queens on road concrete that the same relations do not hold for cored or sawed specimens of job concrete. Starting from the observation that 4-in. cubes cut from a 1:3:6 pavement base frequently show an average compressive strength of 2000 lb. per sq. in., whereas molded cylinders of the same batches rarely exceed 1400 lb. per sq. in., cubes were sawed from slabs cast after the manner of pavement foundation and compared with 6 by 12-in. and 8 by 16-in. cylinders molded from the same batches. The results for $\frac{3}{4}$-in. silicate gravel, $\frac{3}{4}$-in. limestone and $1\frac{1}{2}$-in. limestone coarse aggregate, all with Cow Bay sand as fine aggregate in the bulk proportions of 1:3:6 are given in Table I. Mr. Butterfield.

One condition which we have frequently encountered in road concrete is again reflected in the tables. The compressive strength of cubes is greater when the load is applied in the same direction as the direction of the compaction of the concrete than when the load is applied in a direction at right angles thereto. The difference is greater with some aggregates, particularly gravel, than it is with others. In the present series of about 200 tests, the average strength is 12.5 per cent greater in the direction in which the concrete was compacted than in a direction at right angles thereto.

A point of difference between Mr. Gonnerman's results and ours is in the value for the compressive strength of 6 by 12-in. and 8 by 16-in. cylinders of 1:7 and 1:3:6 concrete. In Table IV, Mr. Gonnerman gives the compressive strength as:

Mix	Water Ratio	6 by 12-in. Cylinders	8 by 16-in. Cylinders
1:7.....	1.24	2160 lb. per sq. in.	1790 lb. per sq. in.

whereas we find for a water ratio of 1.22 only 763 lb. per sq. in. for 6 by 12-in. cylinders and 1048 lb. per sq. in. for 8 by 16-in. cylinders and an average of 814 lb. per sq. in. for all 6 by 12-in. cylinders and 856 lb. per sq. in. for 8 by 16-in. cylinders, or an average of 835 lb. per sq. in. for all cylinders, a figure which is more in accord with

[1] Chemist, Office of President, Borough of Queens, Long Island City, N. Y.

Mr. Butterfield. J. C. Pearson's figure[1] of 820 lb. per sq. in. for 1:3:6 concrete with a water ratio of 1.45. Our mixes were all workable and were of two consistencies as judged by the slump test. One consistency was about that which is used in good concrete-base practice and the other was decidedly sloppy such as would only be used in exceptionally bad work.

TABLE I.—COMPRESSIVE STRENGTH OF CYLINDERS AND SAWED CUBES OF 1:3:6 CONCRETE, LB. PER SQ. IN.

Water-Cement Ratio	Cylinders		Cubes					
	6 by 12-in.	8 by 16-in.	4-in.		6-in.		8-in.	
			a	*b*	*a*	*b*	*a*	*b*
				¾-IN. GRAVEL				
1.25	....	....	....	....	1555	1248	....	....
1.35	842	....	....	....	....	....	1446	1392
1.38	....	897	....	....	....	....	....	....
1.42	....	....	....	....	1092	....	1465	1245
1.44	....	....	1240	952	....	....	....	....
1.47	....	....	....	....	....	1069	1304	1230
1.51	....	751	1122	868	....	....	....	....
Average	821	824	1181	910	1329	1159	1405	1289
				¾-IN. LIMESTONE				
1.32	918	....	....	....	....	....	....	....
1.35	....	....	1690	1683	....	....	....	....
1.47	742	....	1611	1451	....	....	....	....
Average	830	....	1651	1567	....	....	....	....
				1½-IN. LIMESTONE				
1.15	....	....	....	....	....	1685	....	....
1.18	....	909	....	....	1673	....	....	....
1.22	763	1048	2206	1735	....	....	2042	1869
1.25	....	....	....	....	....	1516	....	....
1.28	817	889	1897	1713	1848	....	2039	1644
1.32	....	688	....	....	....	....	....	....
Average	790	887	2052	1724	1761	1601	2041	1757

[a] Load applied in the same direction as the direction in which the concrete was compacted.
[b] Load applied at right angles to the direction of the compaction of the concrete.

The two consistencies as judged by the slump test permit of considerable variation of the water-cement ratio for each consistency; the effect upon the results, however, is not so great as one would expect. All slabs and the corresponding sets of cylinders were proportioned, cured and tested under the same conditions as nearly as possible. The main difference is in the method of placing and compacting cylinders

[1] J. C. Pearson, "Economic Value of Admixtures," *Proceedings*, Am. Concrete Inst., Vol. 20, p. 318, Table I (1924).

by A.S.T.M. method[1] and slabs 6, 8 and 10 in. in thickness. The differences in the strength relations are sufficient to raise a question as to the applicability to job concrete of correction factors obtained on molded laboratory specimens. Mr. Butterfield.

SUMMARY OF STRENGTH RELATIONS FOR 1:3:6 CONCRETE

Aggregate	Cubes, parallel / Cubes, perpendicular	Cubes, parallel / Cylinders	Cubes, perpendicular / Cylinders	All Cubes / Cylinders
¾-in. Gravel	1.17	1.59	1.36	1.47
¾-in. Limestone	1.05	1.98	1.88	1.94
1½-in. Limestone	1.15	2.33	2.02	2.17
Average	1.13	1.97	1.75	1.86

"Cubes, parallel" denote cubes on which the compressive load was applied in the same direction as the direction of compaction of the concrete, and "Cubes, perpendicular" are cubes on which the load was applied in a direction at right angles to the direction of compaction of the concrete. From the results it appears that for slabs of lean concrete such as 1:3:6 there are other factors to be considered besides the water-cement ratio, the consistency and the geometrical form of the test specimens. In all probability the size and shape of the particles of coarse aggregate, the space relations between the particles of coarse aggregate and the mortar in concrete slabs, and the degree of compaction have some influence on the compressive strength. However that may be, it is interesting to note that the strength of job concrete and concrete slabs fashioned after the manner of job concrete is greater than that of the same concrete molded in 6 by 12-in. or 8 by 16-in. cylinders by the standard method, with or without the application of prevailing correction factors for the ratio of height to diameter or width of the test specimen.

MR. H. F. GONNERMAN (*author's closure by letter*).—The data on tests of molded cylinders and sawed cubes of 1:3:6 concrete presented by Mr. Butterfield show a different relation between the strength of cylinders and cubes than that found by the author. It should be noted, however, that in the author's tests comparisons are based on specimens molded in metal or metal-lined molds, compacted and cured in the same manner and loaded parallel to the direction of compacting in the mold. Part of the difference in the results of the author and those found by Mr. Butterfield may be attributed to differences Mr. Gonnerman.

[1] A.S.T.M. Tentative Methods of Making Compression Tests of Concrete (C 39-21 T), *Proceedings* Am. Soc. Testing Mats., Vol. 21, p. 379 (1921).

Mr. Gonnerman.

in method of compacting the molded cylinders and the slabs from which cubes were cut; probably differences in curing also influenced the results although the method of curing was not stated by Mr. Butterfield. If 6 by 12-in. or 8 by 16-in. cylinders had been cut from the slabs for comparison with the sawed cubes, no doubt relations more in agreement with those of the author would have been found, for then the conditions of test for both cylinders and cubes would have been more nearly equal. It is of interest to note in this connection that in field tests of concrete carried out at Camden and Newark, N. J.,[1] 28-day and 3-month tests of $4\frac{1}{2}$ by 8-in. cores drilled from concrete slabs and of 6 by 12-in. cylinders molded from the same batch of concrete as the slabs showed, in general, about the same strengths for the cores as for the molded cylinders.

Mr. Butterfield finds that the compressive strength of concrete cubes is greater when the load is applied parallel to, than when it is applied perpendicular to the direction of compacting. Tests bearing on this question made in our laboratory on 2-in. cubes of neat cement and 1:3 standard sand mortar gave the following results:

Age at Test, days	Compressive Strength, lb. per sq. in.								
	Neat Cement						1:3 Standard Ottawa Sand Mortar		
	Normal Consistency (Water Ratio 0.35)			1.80 Consistency (Water Ratio 0.62)			Normal Consistency (Water Ratio 0.62)		
	[a]	[b]	Ratio [b] to [a], per cent	[a]	[b]	Ratio [b] to [a], per cent	[a]	[b]	Ratio [b] to [a], per cent
1	2 970	2 950	99	350	370	106	510	380	74
3	5 750	5 800	101	1 010	1 020	101	1 250	1 000	80
7	7 940	8 330	105	1 720	1 630	95	1 700	1 520	90
28	10 190	10 790	106	3 520	3 300	94	2 770	2 500	90
	Average.....		103			99			84

[a] Load applied parallel to direction of compacting.
[b] Load applied perpendicular to direction of compacting.

The above data show but little difference in compressive strength of neat cement whether the cube is loaded perpendicular to the direction of compacting or parallel to it. The results for the 1:3 standard sand mortar confirm those found by Mr. Butterfield for concrete, in that lower strength (16 per cent lower) was obtained for the cubes loaded perpendicular to the direction of compacting. On the other hand, tests reported by Otto Graf[2] on concrete cubes of 1:2:3 mix

[1] "Report on Field Tests of Concrete," by W. A. Slater and Stanton Walker, *Proceedings*, Am. Soc. Civil Engrs., January, 1925, p. 29.

[2] "Die Druckfestigkeit von Zementmörtel, Beton, Eisenbeton und Mauerwerk," by Otto Graf, p. 5. Published by K. Wittwer, Stuttgart, 1921.

of plastic consistency made from gravel and crushed basalt coarse aggregate showed from 2 to 9 per cent higher compressive strength at 45 days when loaded perpendicular to the direction of tamping than when loaded parallel to the direction of tamping. Graf found that concrete cubes made from flat, smooth gravel particles when loaded perpendicular to the direction of tamping gave 1 per cent lower compressive strength than when loaded parallel to the direction of tamping; he states that for very coarse gravel the effect of shape and smoothness of surface of particles may be expected to be more pronounced. Mr. Gonnerman

It appears that the existing data on this question are conflicting and that further tests are necessary to determine definitely the relation which exists between the strength of concrete cubes loaded perpendicular and parallel to the direction of compacting.

Mr. Butterfield points out that for 1:3:6 concrete of water-ratio 1.22 he obtained an average compressive strength of 835 lb. per sq. in. in tests of 6 by 12-in. and 8 by 16-in. cylinders whereas in the author's tests the average strength of 6 by 12-in. and 8 by 16-in. cylinders of 1:7 concrete, water-ratio 1.24, was 1975 lb. per sq. in. at 28 days. The latter strength is somewhat higher and Mr. Butterfield's value is somewhat lower than is usually obtained for this water-ratio. This discrepancy in results is probably due to differences in materials, in manipulation, curing, etc.

STUDIES OF BOND BETWEEN CONCRETE AND STEEL

By Duff A. Abrams[1]

Synopsis

The bond between concrete and steel is an important element in reinforced concrete construction. The investigation described in this paper was undertaken for the purpose of determining the relation between bond and the factors which are known to govern the compressive strength of concrete. Bond tests were made by applying a pull on one end of 1-in. plain round steel bars embedded axially in 8 by 8-in. concrete cylinders. The concrete covered a wide range in size and grading of aggregate, quantity of mixing water and cement, and was tested at ages of 7 days to 1 year. Hydrated lime and crude oil were used as admixtures in a few tests. There were 735 pull-out bond tests, and 735 parallel compression tests on 6 by 12-in. cylinders.

The data presented show the load-slip relations for pull-out tests; the relation between bond and compressive strength; the effect of grading of aggregate, quantity of cement, consistency of concrete and age at test, upon the bond and compressive strength; and the effect upon the bond of oil and hydrated lime as an admixture.

The data indicate that the use of 4 per cent of the 28-day compressive strength of concrete as working stress in bond for plain bars as specified by the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, is justified, in that it gives a factor of safety of about 2½ to 3 against slip.

Introduction

The working together of concrete and steel in a reinforced concrete member or structure depends on the bond which is developed at the surface of the reinforcement. An investigation carried out by the author at the University of Illinois[2] showed the nature of bond resistance and indicated that it was independent of the size of bar or length of embedment, but depended essentially on the quality of the concrete and the condition of the surface of the bar. Those tests indicated a fairly constant relation between bond resistance for plain rolled bars and the compressive strength of concrete; however, at that time the fundamentals of concrete mixtures were not fully appreciated;[3] consequently it seemed desirable to carry out further bond tests in the light of more recent studies of the factors which control concrete strength.

[1] Professor in Charge, Structural Materials Research Laboratory, Lewis Institute, Chicago.

[2] "Tests of Bond Between Concrete and Steel," *Bulletin No. 71*, University of Illinois Engineering Experiment Station (1913).

[3] D. A. Abrams, "Design of Concrete Mixtures," *Bulletin No. 1*, Structural Materials Research Laboratory (1918).

An important recent report from the Bureau of Standards gives the results of bond tests made by the U. S. Shipping Board in 1918[1] on plain and deformed bars coated with various protective coverings.

The chief purpose of the present investigation was to study bond resistance as influenced by variations in quantity of mixing water (the water-ratio), grading of aggregate, quantity of cement, consistency, age, etc. The tests form a part of the experimental studies of concrete and concrete materials being carried out through the co-operation of Lewis Institute and the Portland Cement Association at the Structural Materials Research Laboratory, Chicago.

MATERIALS AND TEST PIECES

Pull-out bond tests were made by applying tension to one end of a 1-in. plain round steel bar 18 in. long embedded axially in an 8 by 8-in. concrete cylinder.

Tests were made under the following headings:

1. Size of aggregate (7 sizes of sand and pebbles from 0 – No. 28 to 0 – 1½ in.).
2. Grading of aggregate (7 gradings of 0 – 1½ in.).
3. Mixture (1:7 concrete to neat cement).
4. Consistency of concrete (relative consistencies 0.90 to 1.50).
5. Age at test (7 days to 1 year).
6. Hydrated lime admixture (5 to 50 per cent of cement replaced).
7. Crude oil admixture (2 to 33 per cent of mixing water replaced).

Parallel compression tests were made on 6 by 12-in. concrete cylinders. Of each type (bond and compression), 735 specimens were tested.

The mix was expressed in terms of 1 volume of cement to a given number of volumes of dry and rodded aggregate mixed as used. Cement was considered to weigh 94 lb. per cu. ft. Concrete having a relative consistency of 1.00 will slump about ½ to 1 in. in the test with the 4 by 8 by 12-in. truncated cone. A relative consistency of 1.10 requires 10 per cent more water, etc.

The cement was a mixture of equal parts of 4 brands of portland cement purchased in Chicago.

The aggregates consisted of calcareous sand and pebbles from the Elgin, Ill., pit of the Chicago Gravel Co. The size and grading of

[1] Slater, Richart and Scofield, "Tests of Bond Resistance Between Concrete and Steel," *Technologic Paper No. 173*, U. S. Bureau of Standards (1920).

the aggregate was measured by the fineness modulus (m). This function is the sum of the percentages in the sieve analysis divided by 100 when the sieve analysis is expressed in percentages coarser than each of the sieves given below. Each sieve has a clear opening double that of the next smaller. Following is the sieve analysis of

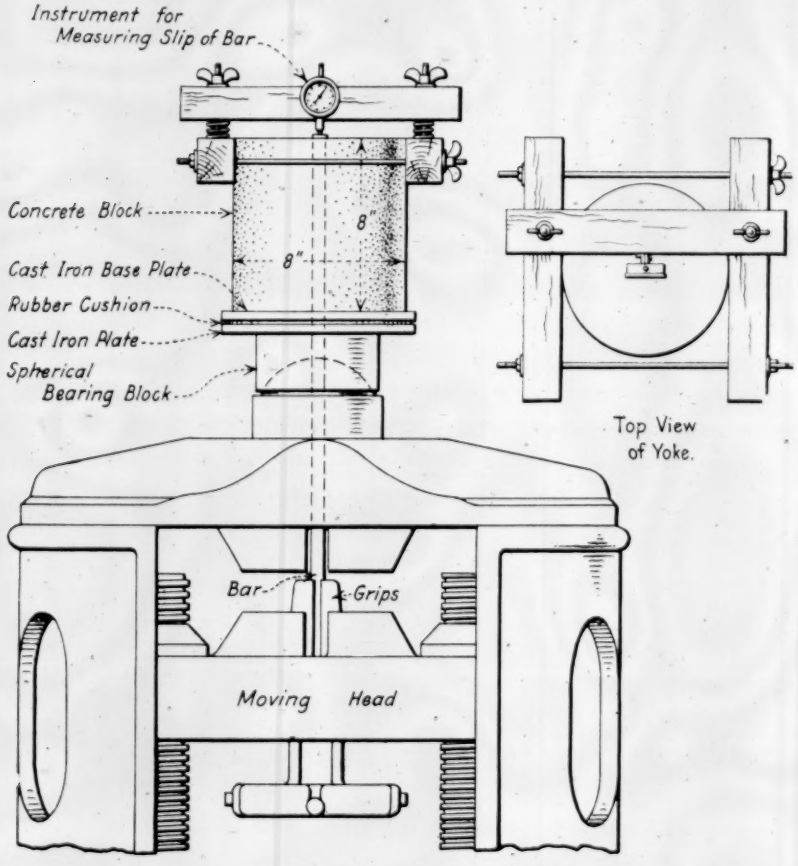

FIG. 1.—Apparatus for Pull-out Tests of Bond Between Concrete and Steel.

the 0 – 1½-in. aggregate (fineness modulus 5.75) which was used in most of the tests:

AMOUNTS COARSER THAN EACH SIEVE, PER CENT BY WEIGHT									FINENESS
No. 100	No. 48	No. 28	No. 14	No. 8	No. 4	⅜-IN.	¾-IN.	1½-IN.	MODULUS
99	98	87	80	74	69	48	20	0	5.75

The mixing water was from the Chicago water supply from Lake Michigan.

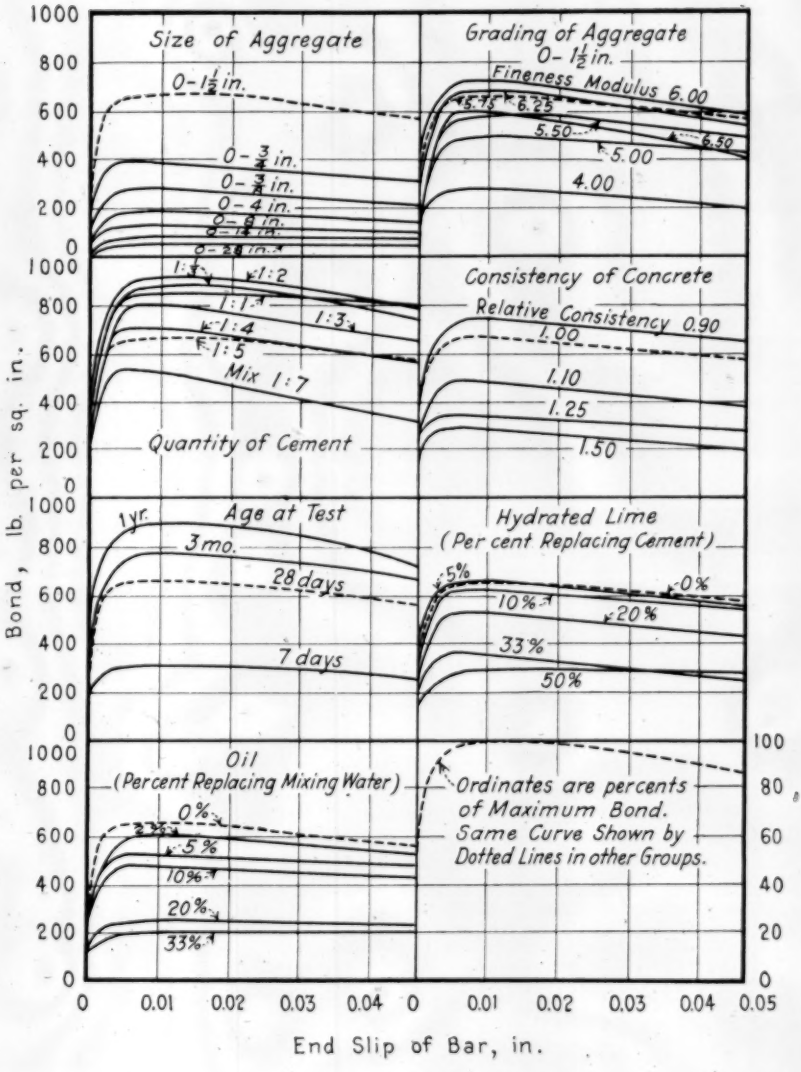

FIG. 2.—Load-Slip Curves.

Pull-out tests of 1-in. plain round bars embedded 8 in.

Age at test, 28 days, unless otherwise noted.

The principal variable is indicated for each group; unless otherwise noted, the concrete was 1:5 mix, relative consistency 1.00, sand and pebble aggregate graded 0 – 1½ in. (fineness modulus 5.75). Cured in a damp place and tested at age of 28 days. Concrete of this quality is common to each group—shown by dotted lines.

The concrete was mixed by hand with a blunted trowel in metal pans in batches of sufficient size (about $\frac{1}{5}$ cu. ft.) to make one test piece. All specimens were cured in a moist room until the day of test.

Plain bars of 1-in. diameter were used since they were of typical surface and large enough to prevent over-stressing the steel.

In general, each value is the average of five tests made on different days. Certain "key" values were repeated at different points in the series until a total of 15 tests were obtained, which provided a dependable basis for comparison.

Most of the tests were made at 28 days on 1:5 concrete, aggregate graded 0 – $1\frac{1}{2}$ in. and water-ratio 0.88, or about 6.6 gal. of mixing

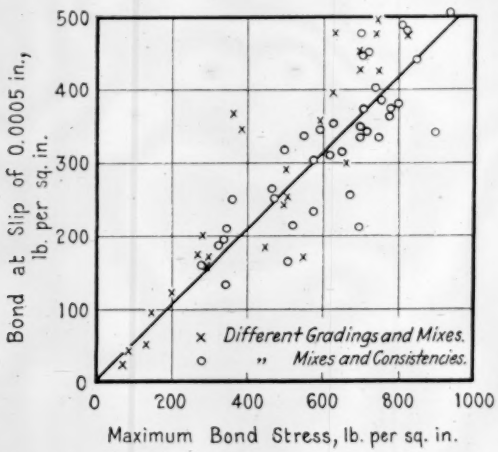

FIG. 3.—Relation of Bond at 0.0005-in. End Slip and Maximum Bond.

Maximum bond values from Tables I, II and III.

Includes mixes ranging from 1:7 to neat, relative consistencies 0.90 to 1.50, aggregate gradings from 0 – No. 28 to 0 – $1\frac{1}{2}$ in., and ages 7 days to 1 year.

water per sack of cement. This concrete gave at 28 days an average bond resistance of 680 lb. per sq. in. and a compressive strength of 2730 lb.

The pull-out tests were made by the method used by the writer at the University of Illinois, illustrated in Fig. 1. The spherical bearing block avoided side pull. The slip of the bar during loading was measured by a 0.0001-in. Ames dial supported by a frame-work, carried by the concrete specimen, with the moving plunger of the dial in contact with the free end of the test bar. Load-slip readings were taken in all pull-out tests; in general the first appreciable end slip was recorded at about 0.0002 in.; readings were continued to a slip of 0.1 in.

DISCUSSION OF TESTS

Load-Slip Relation for Pull-Out Tests.—A steel bar embedded in concrete offers considerable resistance against being pulled out. Two distinct stages may be recognized:

1. Anti-slip bond or adhesion; and
2. Sliding friction.

Anti-slip bond is a characteristic of the nature of the cement mortar and steel and is exhibited even by highly polished bars. Sliding friction

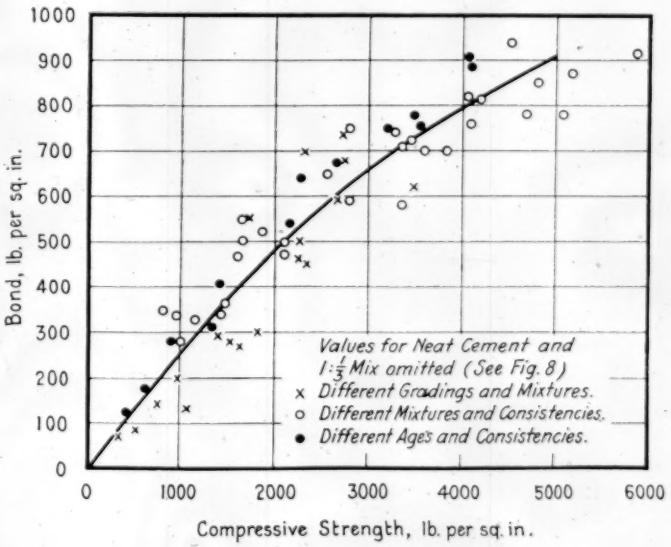

FIG. 4.—Relation of Bond and Compressive Strength of Concrete.

Bond tests on 1-in. plain round bars embedded 8 in.
Compression tests of 6 by 12-in. cylinders.
Mixes ranged from 1:7 to neat, consistencies 0.90 to 1.50, aggregate gradings 0 – No. 28 to 0 – 1½ in., and ages 7 days to 1 year.
Data from Tables I, II and III. Same values in Figs. 6, 7, 8 and 9.

results from the inequalities in the surface of mill-steel bars or shapes, and the conformity of the surrounding concrete.

Fig. 2 gives typical load-slip curves. The 1:5 concrete curve, which is common to all groups, is replotted in the lower right corner with the percentage of maximum bond as ordinates.

Fig. 3 gives the relation between bond at end slip of 0.0005 in. and the maximum bond, referred to hereafter as "bond." The average of all tests shows the bond at 0.0005 in. end slip to be about 52 per cent of the ultimate, with a range of 40 to 60 per cent. The

TABLE I.—EFFECT OF SIZE AND GRADING OF AGGREGATE ON THE BOND AND COMPRESSIVE STRENGTH OF CONCRETE.

Bond—Pull-out tests on 1-in. plain round steel bars embedded 8 in. in 8 by 8-in. concrete cylinders.
Compression—6 by 12-in. cylinders.
Aggregate: sand and pebbles graded as shown.
Cement: a mixture of 4 brands of portland cement purchased in Chicago.
Concrete: hand-mixed in batches of about $\frac{1}{5}$ cu. ft.
Relative consistency, 1.00.
Age at test, 28 days.
Cured in damp condition until tested.
Each value is the average of 5 tests, unless otherwise noted.
Data plotted in Figs. 3 to 7.

Aggregate		1:5 Concrete			1:3 Concrete		
Size	Fineness Modulus	Water-Ratio	Bond, lb. per sq. in.	Compressive Strength, lb. per sq. in.	Water-Ratio	Bond, lb. per sq. in.	Compressive Strength, lb. per sq. in.
0 - No. 28	1.20	1.42	70	320	0.99	130	1020
0 - No. 14	2.25	1.29	80	500	0.92	270	1620
0 - No. 8	2.65	1.24	140	740	0.89	300	1810
0 - No. 4	3.00	1.20	200	960	0.87	450	2330
0 - $\frac{3}{8}$ in.	4.00	1.08	290	1370	0.80	620	3460
0 - $\frac{3}{4}$ in.	5.00	0.96	460	2230	0.72	750	4020
0 - 1$\frac{1}{2}$ in.	5.75	0.88	680[a]	2730[a]	0.67	820[b]	4040[b]
0 - 1$\frac{1}{2}$ in.	4.00	1.08	280	1520	....	....	....
0 - 1$\frac{1}{2}$ in.	5.00	0.96	500	2250	....	....	....
0 - 1$\frac{1}{2}$ in.	5.50	0.90	590	2650	....	...	....
0 - 1$\frac{1}{2}$ in.	5.75	0.88	680[a]	2730[a]	0.67	820[b]	4040[b]
0 - 1$\frac{1}{2}$ in.	6.00	0.84	740	2730	....	...	....
0 - 1$\frac{1}{2}$ in.	6.25	0.82	700	2300	....	...	....
0 - 1$\frac{1}{2}$ in.	6.50	0.78	550	1680	....	...	....

[a] Average of 15 tests common to both groups in this table; the same values occur in Tables II to V.
[b] Average of 10 tests common to both groups in this table; the same values occur in Table II.

TABLE II.—EFFECT OF MIX AND CONSISTENCY OF CONCRETE.

Aggregate: sand and pebbles graded 0 - 1$\frac{1}{2}$ in. ($m = 5.75$).
Age at test, 28 days.
See Table I for more complete notes on tests.
Data plotted in Figs. 3, 4, 5, 8, and 9.

Mix by Volume	Water-Ratio for Relative Consistency 1.00[a]	Bond and Compressive Strength, lb. per sq. in., for Concrete of Different Relative Consistencies									
		0.90		1.00		1.10		1.25		1.50	
		Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.
1:7	1.08	470	2070	550	1690	340	1400	340	940	350	770
1:5	0.88	740	3260	680[b]	2730[b]	500	2090	360	1450	280	990
1:4	0.77	810	4180	710	3340	590	2760	500	1640	330	1130
1:3	0.67	780	4690	820[c]	4040[c]	700	3570	650	2520	470	1570
1:2	0.57	850	4780	940	4500	760	4070	750	2790	520	1840
1:1	0.46	910	5850	870	5150	780	5070	700	3830	720	3440
1:$\frac{1}{2}$	0.39	800	6380	900	6680	680	5720	670	5380	620	4120
Neat	0.36	580	3340	710	6960	650	6820	580	6610	510	4930

[a] Water-ratio for other consistencies may be calculated by multiplying the values given by "relative consistency."
[b] Average of 15 tests; the same values occur in Tables I to V.
[c] Average of 10 tests; the same values occur in Table I.

University of Illinois tests gave a ratio of 70 per cent, which in the present tests corresponds to an end slip of about 0.0010 in.

Relation Between Bond and Compressive Strength.—Fig. 4 gives the following average ratios for the relation between bond and compressive strength for different mixes, consistencies, aggregate gradings and ages from Tables I, II and III:

COMPRESSIVE STRENGTH OF CONCRETE, LB. PER SQ. IN.	BOND, PER CENT OF COMPRESSIVE STRENGTH
1000	28
2000	25
3000	22
4000	20

The data for the 1:$\frac{1}{3}$ and neat cement mixes have been omitted from this diagram; these tests are discussed below. For usual concrete mixtures the bond on plain steel bars is about 24 per cent of the

TABLE III.—EFFECT OF CONSISTENCY OF CONCRETE.

Mix, 1:5 by volume.
Aggregate: sand and pebbles graded 0 – 1½ in. (m=5.75).
See Table I for more complete notes on tests.
Data plotted in Figs. 9 and 10.

Relative Consistency	Water-Ratio	Bond and Compressive Strength, lb. per sq. in.							
		7 days		28 days		3 months		1 year	
		Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.
0.90	0.79	410	1400	740	3260	890	4080	950	4520
1.00	0.88	310	1310	680[a]	2730[a]	760	3550	910	4020
1.10	0.96	280	860	500	2090	750	3160	780	3400
1.25	1.09	180	600	360	1450	640	2250	680	2640
1.50	1.31	120	380	280	990	500	1620	540	2110

[a] Average of 15 tests; the same values occur in Tables I to V.

compressive strength. The University of Illinois tests gave the same value. The ratio decreased slightly for concrete of higher strengths.

The factors which produced high compressive strengths also produced high bond resistance. Other tests have shown that the same statement applies in general to resistance to wear, modulus of elasticity, impermeability and resistance to destructive agencies such as weather, sea and sulfate waters, etc. It is particularly striking that curves showing the relation between modulus of elasticity[1] and compressive strength have a shape similar to Fig. 4. The significance of this intimate relation between bond and modulus of elasticity is not clear.

[1] See "Modulus of Elasticity of Concrete," by Stanton Walker, *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part II, p. 510 (1919).

Water-Ratio-Bond Relation.—The influence of the quantity of mixing water as expressed by the "water-ratio" on both bond and compressive strength is shown in Fig. 5. This curve shows that bond is influenced by changes in water-ratio in exactly the same way as the compressive strength. The water-ratio is defined as the ratio

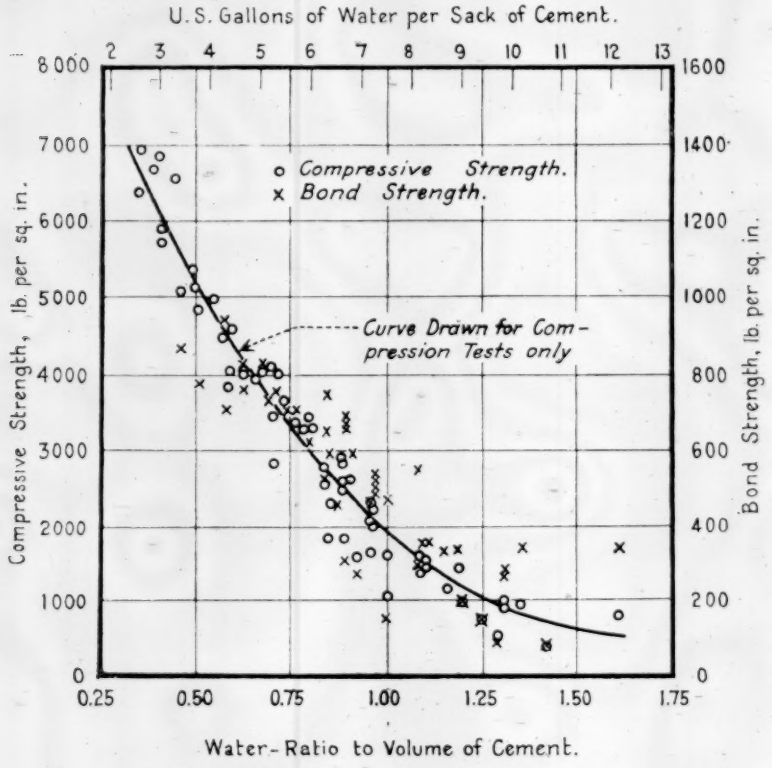

FIG. 5.—Water-Ratio-Strength Relation for Concrete.

28-day tests.

Includes mixes ranging from 1:7 to neat, relative consistencies 1.00 to 1.50 and aggregate grading from 0 – No. 28 to 0 – 1½ in.

Bond values for neat cement and 1: ⅓ mix omitted.

All values for relative consistency 0.90 and fineness modulus 6.25 and 6.50 omitted.

Data from Tables I, II and III. Same values in Figs. 6, 7, 8 and 9.

of the volume of water to volume of cement in the batch, considering 94 lb. of cement as 1 cu. ft. All the 28-day tests in Tables I to III were included except relative consistency 0.90 and fineness moduli of aggregate higher than 6.00, which were omitted since they were not sufficiently workable to permit satisfactory placing. In addition, the value for neat cement and 1:⅓ mix were omitted in plotting the

bond tests on this curve, for reasons discussed below. This curve is typical of those obtained in many other series of tests carried out in this Laboratory.

Effect of Grading of Aggregate.—Bond and compressive strength of concrete from sand and pebble aggregate of different size and grading are shown in Table I and Figs. 6 and 7. These data show that both bond and compressive strength increased with fineness modulus of aggregate so long as the grading was not too coarse for proper workability. The fineness modulus is a measure of the size

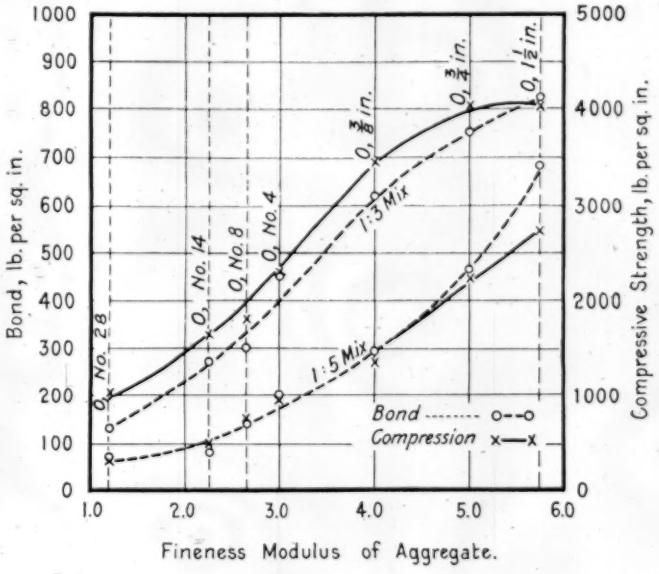

FIG. 6.—Effect of Size of Aggregate on Bond and Compressive Strength.
Age at test 28 days. Data from Table I. Compare Fig. 7.

and grading of aggregate; a low value indicates aggregate of small size or one containing a large proportion of fine.

Fig. 6 shows tests of different sizes of aggregate. For the 1:5 mix (same quantity of cement in all mixtures) the compressive strength varied from 320 lb. per sq. in. for 0 – No. 28 sand to 2730 lb. per sq. in. for aggregate graded up to 1½ in.; bond varied from 70 to 680 lb. per sq. in. for the same conditions. This is only another way of saying that the quality of concrete increases as the quantity of mixing water is reduced since, in a concrete of uniform workability, a coarse well-graded aggregate requires less mixing water than one poorly graded. The same tests are included in Figs. 4 and 5.

In Fig. 7 the range in fineness modulus was obtained by mixing different proportions of sand and pebbles. The fineness modulus of 4.0 for aggregate graded 0 – 1½ in. consisted of 75 per cent sand and 25 per cent pebbles; a fineness modulus of 6.0, 25 per cent sand and 75 per cent pebbles. The bond was relatively higher than the compressive strength for the coarser gradings. For fineness moduli higher than 6.0, the aggregates were too coarse to produce a workable mix and consequently both bond and compression dropped off.

Effect of Quantity of Cement.—Bond and compressive strength of concrete of different quantities of cement (mixtures ranging from

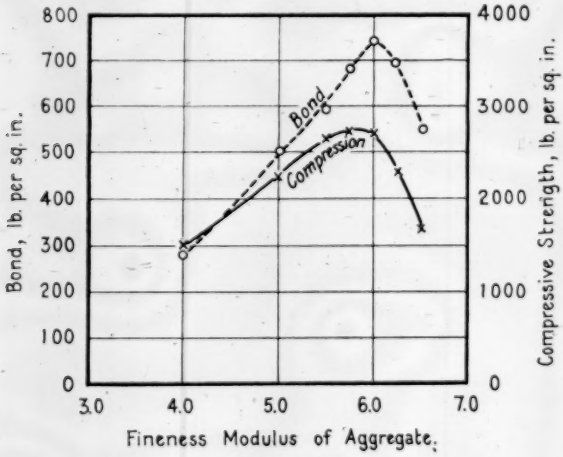

FIG. 7.—Effect of Grading of Aggregate.

Mix 1:5. Relative consistency, 1.00. Aggregate graded 0 – 1½ in.
Age at test, 28 days.
Data from Table I. Compare Fig. 6.

1:7 to neat) are shown in Table II and Fig. 8. The data for relative consistencies 1.00 and 1.50 only are plotted; including the three other consistencies, which gave similar curves, would only confuse the diagram. The quantity of cement affected the bond strength in about the same manner as it affected the compressive strength for mixtures leaner than 1:1.

The bond strengths for the 1:⅓ and neat cement mixes are lower than for the 1:1 mix. This may be due to the volume changes of the concrete during hardening which may tend to destroy the bond.

The ratio of bond to compression ranged from about 30 per cent for the 1:7 mix to about 17 per cent for the 1:1 mix. For the usual

range in mixes (1:5 to 1:4) the ratio was 24 per cent; for the 1:$\frac{1}{3}$ and neat cement mixes, bond was about 10 per cent of the compression.

Effect of Consistency of Concrete.—Data of tests of concrete of consistencies ranging from dry to very wet (Table II) are plotted in Fig. 9. These curves show that increasing the mixing water rapidly

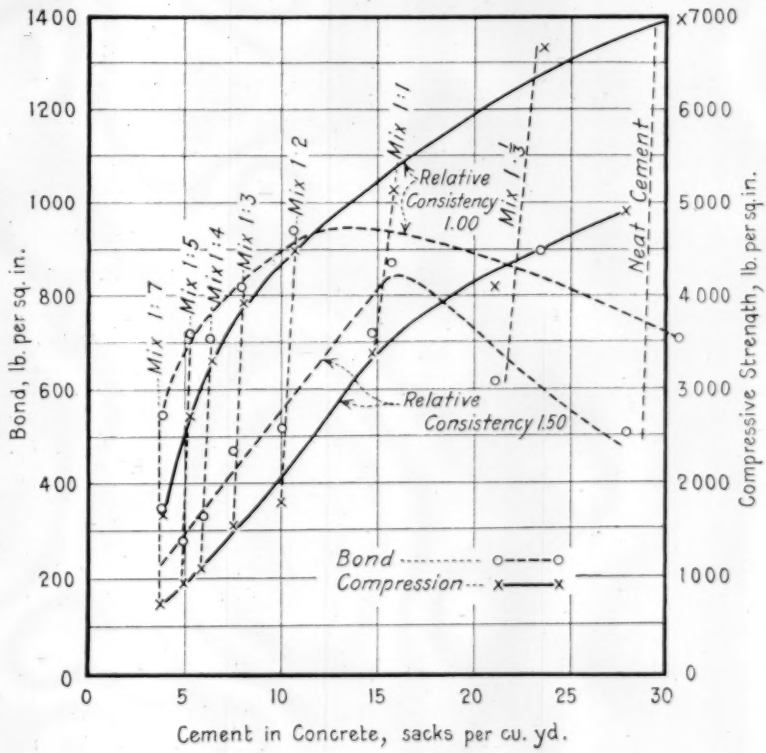

FIG. 8.—Effect of Quantity of Cement.

Mix, 1:5. Aggregate, 0 – 1½ in.
Age at test, 28 days.
Data from Table II. Relative consistencies 0.90, 1.10 and 1.25 have been omitted.

decreased the bond and compressive strength. For the 1:5 mix the compressive strength at 28 days for relative consistency of 1.00 was 2730 lb. per sq. in., while for a relative consistency of 1.50 (50 per cent more water) the strength was only 900 lb. per sq. in., a reduction of 67 per cent.

It will be noted that the slopes of the bond curves are flatter than those for compression; increasing the relative consistency of the con-

crete from 1.00 to 1.50 decreased the bond from 680 to 280 lb. per sq. in., a reduction of 59 per cent. The same phenomenon is shown in Table II. For the 1:5 mix, the ratio of bond to compression increased from about 25 per cent for relative consistency 1.00 to about 28 per cent for relative consistency 1.50. In many cases bond for relative

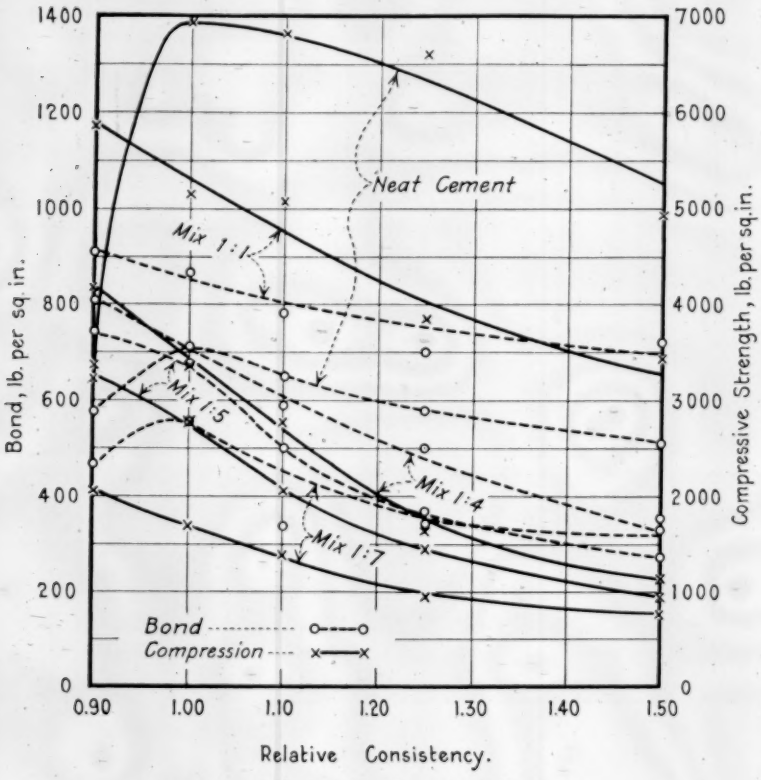

FIG. 9.—Effect of Consistency of Concrete.

Aggregates graded 0 – 1½ in. (m = 5.75). Age at test, 28 days.
Data from Table II.
Compare Figs. 6, 7 and 8 where water-ratio was varied due to grading of aggregate and mix.

consistency 0.90 was lower than for 1.00; the former concrete was too dry to permit proper placing. Compare Figs. 6, 7 and 8, where the water-ratio of the concrete was varied due to grading of aggregate and to quantity of cement.

Effect of Age at Test.—Tests at ages ranging from 7 days to 1 year are given in Tables III, IV and V and Fig. 10. Bond was affected by age in essentially the same manner as compression; both increase

TABLE IV.—CRUDE OIL AS AN ADMIXTURE IN CONCRETE.

Oil added to concrete before final mixing.
Mix 1:5 by volume.
Relative consistency, 1.00.
Aggregate: sand and pebbles graded 0 – 1½ in. (m=5.75).
See Table I for more complete notes on tests.

Mixing Fluid, per cent		Water-Ratio	Liquid-Cement Ratio	Bond and Compressive Strength, lb. per sq. in.							
				7 days		28 days		3 months		1 year	
Oil	Water			Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.
0	100	0.88	0.88	310	1310	680[a]	2730[a]	760	3550	910	4020
2	98	0.86	0.88	350	1250	620	2530	870	3950	910	3750
5	95	0.84	0.89	340	1330	550	2640	780	3490	860	3880
10	90	0.82	0.91	260	1220	520	2240	690	3010	740	3420
20	80	0.75	0.94	160	1250	300	2300	400	2760	480	3160
33	67	0.68	1.01	150	1430	220	2520	300	3080	350	3280

[a] Average of 15 tests; the same values occur in Tables I to V.

TABLE V.—HYDRATED LIME AS AN ADMIXTURE IN CONCRETE.

Mix 1:5 by volume (1 vol. of cement and hydrated lime to 5 volumes of aggregate).
Relative consistency, 1.00.
Aggregate: sand and pebbles graded 0 – 1½ in. (m=5.75).
See Table I for more complete notes on tests.
Data plotted in Fig. 11.

Hydrated Lime, per cent	Portland Cement, per cent	Water, Ratio to Volume of Cement	Bond and Compressive Strength, lb. per sq. in.							
			7 days		28 days		3 months		1 year	
			Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.
0	100	0.88	310	1310	680[a]	2730[a]	760	3550	910	4020
5	95	0.91	360	1310	660	2410	740	3730	940	4120
10	90	0.96	320	1230	630	2660	710	3530	910	4000
20	80	1.08	310	1120	510	2280	720	2950	860	3800
33	67	1.30	170	810	370	1720	450	2260	620	3080
50	50	1.72	150	520	280	1030	460	1470	440	1810

[a] Average of 15 tests; the same values occur in Tables I to V.

TABLE VI.—HYDRATED LIME AS AN ADMIXTURE IN NEAT CEMENT PASTE.

Relative consistency, 1.00.
See Table I for more complete notes on tests.

Hydrated Lime, per cent	Portland Cement, per cent	Water, Ratio to Volume of Cement for Relative Consistency 1.00[a]	Bond and Compressive Strength, lb. per sq. in. for Concrete of Different Relative Consistencies									
			0.90		1.00		1.10		1.25		1.50	
			Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.	Bond	Compr.
0	100	0.36	580	3340	710	6960	650	6820	580	6610	510	4930
5	95	0.37	390	3560	630	7210	710	5240	590	5590	470	5650
10	90	0.39	190	3700	610	6090	640	5960	540	6070	420	5150
20	80	0.44	290	1420	450	4850	510	5110	450	4760	340	4000
33	67	0.53	45	880	310	1770	360	3820	330	3600	240	3210
50	50	0.71	30	270	130	290	220	2460	220	2180	180	1740

[a] To calculate water-ratio for other consistencies, multiply the value given in this column by "relative consistency."

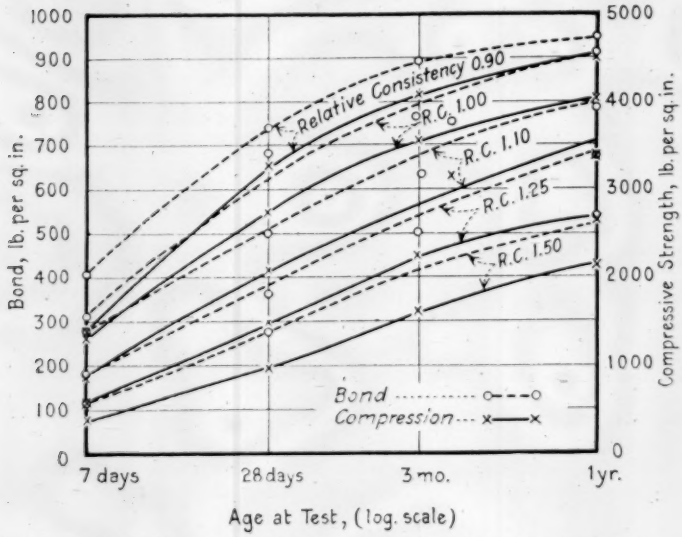

FIG. 10.—Effect of Age of Concrete on Bond and Compression.

Mix 1:5. Aggregates graded 0 – 1½ in. (m = 5.75).
Data from Table III.

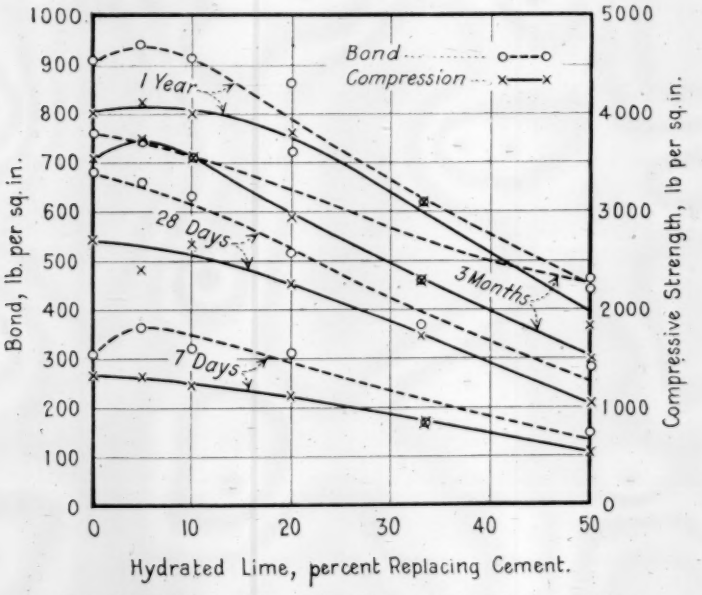

FIG. 11.—Effect of Replacing Cement with Hydrated Lime.

Mix 1:5. Aggregates graded 0 – 1½ in. (m = 5.75).
Data from Table V.

with age so long as the concrete does not dry out; at 1 year bond was 134 per cent of the 28-day value and compression 148 per cent.

Oil as an Admixture in Concrete.—Table IV gives tests in which 2 to 33 per cent of the mixing water was replaced by crude oil. In order to maintain a uniform plasticity it was necessary to increase the combined volume of oil and water from 0.88 for 0 per cent of oil to 1.01 for 33 per cent of oil. At 7 and 28 days the compressive strength was only slightly reduced by the oil, but the bond was materially decreased. The decrease in compressive strength was probably due to the slight increase in quantity of mixing fluid. The decrease in bond resistance was undoubtedly due to the lubricating action of the oil.

Effect of Hydrated Lime.—Tests were made on concrete and neat cement in which from 5 to 50 per cent of the portland cement was replaced by hydrated lime.[1] (See Tables V and VI and Fig. 11.) At 28 days the reduction in compressive strength of the concrete was about 1.2 per cent for each 1 per cent of hydrated lime by volume of cement or 2.0 per cent for each 1 per cent by weight of cement. The reduction of the bond strength was approximately the same.

Table VI gives results of tests of neat cement-lime pastes for relative consistencies ranging from 0.90 to 1.50. In general, both the bond and compressive strength were reduced by the hydrated lime. The bond tests with neat cement were quite erratic, probably due to the volume changes mentioned above.

Working Stresses in Bond.—The Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[2] recommends that bond working stress on plain bars be limited to 4 per cent of the compressive strength of the concrete at 28 days. These tests show that the earliest observed end slip of bar occurs at bond stresses of 10 to 12 per cent of the compressive strength of the concrete. The values recommended by the Joint Committee would, therefore, represent a factor of safety of about $2\frac{1}{2}$ to 3 against first slip of bar.

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[1] For more complete tests see "Effect of Hydrated Lime and Other Powdered Admixtures in Concrete," by Duff A. Abrams, *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920); reprinted as *Bulletin 8*, Structural Materials Research Laboratory.

[2] Standard Specifications for Concrete and Reinforced Concrete, Report of Joint Committee; *Proceedings*, Am. Soc., Testing Mats., Vol. 24, Part I, p. 303 (1924); *Proceedings*, Am. Soc. Civil Engrs., January, 1925.

CONCLUSIONS

Following are the principal conclusions:

1. A steel bar embedded in concrete offers considerable resistance to a pull-out load. Slipping of the bar begins at a bond stress of about 10 to 15 per cent of the compressive strength of the concrete, but considerable additional load is taken before the ultimate bond resistance is reached. For pull-out tests of the type used, 0.0005 in. end slip of bar occurred at 55 to 60 per cent of the maximum bond; for mixtures leaner than 1:1, the maximum bond was about 24 per cent of the compressive strength of the concrete and came at an end slip of about 0.01 in., regardless of the characteristics of the concrete.

2. Bond and compressive strength increased with age of the concrete from 7 days to 1 year. For 1:5 concrete of water-ratio 0.88, the bond at 1 year was 134 per cent of the 28-day value and the compressive strength was 148 per cent.

3. Bond responded to changes in water-ratio of the concrete in much the same way as compressive strength; increase in water-ratio due to use of wetter concrete, less cement, or an excess of fine aggregate, resulted in material reductions in both bond and compressive strength. Other tests have shown that the same statement applies to the modulus of elasticity of concrete, impermeability, resistance to wear and resistance to destructive agencies such as weather, sea and sulfate waters, etc.

4. For mixtures richer than 1:1, the bond fell off, probably due to the greater volume changes during hardening, which is characteristic of such mixtures.

5. The use of 4 per cent of the 28-day compressive strength of concrete as the working stress in bond for plain bars, as specified by the Joint Committee, is justified; this gives a factor of safety of about $2\frac{1}{2}$ to 3 against first slip.

6. The use of crude oil to replace mixing water, in general, caused a reduction in both bond and compressive strength of concrete. Five per cent of oil reduced bond at 28 days about 20 per cent; at 1 year, about 6 per cent; compressive strength at 28 days and 1 year was reduced about 3 per cent.

7. Replacing cement with hydrated lime decreased the compressive strength and bond about 1.2 per cent for each 1 per cent of hydrated lime in terms of volume of cement or about 2.0 per cent for each 1 per cent by weight.

8. The pull-out test of the form used in this investigation is a satisfactory form of specimen for comparative studies of bond.

DISCUSSION

Mr. E. B. Smith[1] (*presented in written form*).—Referring to conclusion No. 7, relative to the effect of the replacement of cement by hydrated lime, the inferred explanation as to the reduced bond strength does not seem to be correct, or at least is not complete. Undoubtedly, if the cementing material is reduced and replaced by a material of less cementing value, the result will be a reduction in strength and bond value. Stone dust, talcum powder, or other more or less inert powders, may produce a similar result if used as a replacement material, and a report showing such relative effects of these materials should have just about as much practical value. Hydrated lime is seldom, if ever, used as a replacement material for cement, but it is often used as an additive material with portland cement, and these strength and bond tests would be of more practical use if they showed the effect of hydrated lime as an added admixture and not as a replacement material. **Mr. Smith.**

The proper explanation of the results in connection with hydrated lime is that when the cement was replaced the resulting cement-sand mixture was leaner, and consequently of less strength in either compression or bond. For instance, referring to Fig. 11, and taking any replacement value, as 30 per cent, the resulting cement-sand mixture will not be 1:5 as given but will be 1:7.1. The bond strength with this 30-per-cent lime replacement value is 430 lb. per sq. in., and the bond strength of a 1:7.1 mixture (from Fig. 8) would extrapolate to about 460 lb. per sq. in.—a rather close check. And the compressive strength would show on this basis in favor of the lime mixture.

I hold no brief for the use of hydrated lime, but do think I have indicated the proper explanation of the results in connection with this material.

Messrs. R. W. Crum[2] and Bert Myers (*presented in written form*).—Mr. Abrams' discussion of the various factors which affect the bond between concrete and steel is of much interest and his proof that this bond follows the same laws as those which affect the strength of concrete is quite conclusive. **Messrs. Crum and Myers.**

A very brief preliminary study of the bond between concrete and steel in connection with concrete pavements made by the Iowa State

[1] Senior Testing Engineer, U. S. Bureau of Public Roads, Washington, D. C.
[2] Engineer of Materials and Tests, Iowa State Highway Commission, Ames, Ia.

Messrs. Crum and Myers.

Highway Commission, and which we expect to complete at a future date, seems to indicate that the bond between concrete and steel is governed by some very interesting factors not touched upon by Mr. Abrams in his paper. The results of this preliminary study seem to indicate that the force required to cause slippage between steel and the concrete in which it is embedded is (above a certain minimum length of embedment) independent of the length of embedment and perhaps it is independent of the area of contact.

While some tests were made using various forms of deformed bars, this discussion will be confined to tests in which plain round bars were used.

One cylinder $4\frac{1}{2}$ in. in diameter and 9 in. high giving an embedment of 9 in. was tested in the same manner that Mr. Abrams tested his specimens except that no spherical bearing block was used. The remaining specimens were so designed that both the reinforcing rod

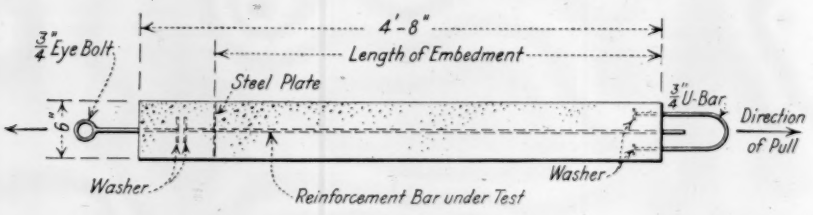

FIG. 1.—Longitudinal Section of Test Specimen.

and the major portion of the concrete in which it was embedded would be in tension during the test. This is a test condition different from that used in Mr. Abrams' experiments. This condition more nearly approximates the conditions imposed by the contraction of a concrete pavement.

These specimens were prisms 6 in. square and 56 in. long. The test rod was embedded in the center of the prism. On one end of the test rod was welded a steel washer which prevented slippage from that end. The other end of the rod extended beyond the end of the prism. In the end of the prism at which the rod protruded was cast a U bolt extending into the concrete about four inches. Steel washers were welded on the ends of the U. The U spanned the free end of the rod. At the end of the prism containing the fixed end of the rod was cast an eye bolt with a washer welded on the end. At some point along the prism between the fixed and exposed ends of the rod was cast a steel plate which divided the concrete transversely. The length of embedment given is the distance from this plate to the end of the

prism at which the rod was exposed. Fig. 1 shows a longitudinal section of the prism. **Messrs. Crum and Myers.**

The pulling force was applied by means of a 200,000-lb. universal testing machine. The U bolt was fastened to the fixed head of the machine and the eye bolt was fastened to the moving head of the testing machine in such a way as to insure an axial pull.

An Ames dial was fastened to the U bolt with its plunger resting against the exposed end of the rod in order to detect when slipping

TABLE I.—RESULTS OF TESTS OF BOND BETWEEN CONCRETE AND PLAIN ROUND REINFORCEMENT RODS.

Specimen	Type of Specimen	Length of Embedment, in.	Diameter of Rod, in.	Total Load at First Slip, lb.	Maximum Load, lb.	Tests of Reinforcement Rod	
						Total Load at Yield Point, lb.	Total Breaking Load, lb.
No. 0	Cylinder	9	$\frac{1}{2}$	7 900	8 700	7 600	11 050
No. 1	Prism	$17\frac{1}{2}$	$\frac{1}{2}$	7 830	7 830	7 600	11 050
No. 3	"	30	$\frac{1}{2}$	7 850	7 850	7 600	11 050
No. 4	"	48	$\frac{1}{2}$	7 500	8 200	7 600	11 050
No. 5	"	48	$\frac{1}{2}$	7 800	8 300	7 600	11 050
No. 11	"	48	$\frac{1}{2}$	7 550	7 950	7 600	11 050
No. 12	"	48	$\frac{1}{2}$	8 500	10 650	7 600	11 050
No. 14	"	48	$\frac{5}{8}$	7 600	18 450	8 900	28 800

TABLE II.

SPECIMEN	SUCCESSIVE LOADS, LB.
No. 11	7550; 7600; 7650; 7700; 7950
No. 12	8500; 8900; 9400; 10650

throughout the length of the bar had occurred. In making the tests the testing machine was operated at a constant speed. The first slip was considered to have occurred when the beam of the testing machine dropped. The application of the load was continued until the bar had slipped throughout its entire length as indicated by the Ames dial. It was found that the rod would take successive increments of load as indicated by the rise of the beam. The maximum load was ordinarily reached just before the bar slipped throughout its length. Table I gives the results of tests of bond of plain round reinforcement bars. Table II gives the observed successive loads at which the beam of the testing machine dropped. These are typical.

These tests seem to indicate:

Messrs. Crum and Myers.

1. That the total load necessary to cause slipping between a plain round steel bar and the embedding concrete is independent of the length of the bar or the area of contact.

2. That the slipping is due to the reduction of the cross-sectional area of the bar.

3. That with a continued application of a tensile stress on the reinforcing bar the bond failure is progressive. That is, with a given increment of load the bond is broken only for a short part of the length of the bar. This section of the bar which is released from true bond with the concrete is then free to elongate under the applied load. As further loading is continued, the stress is transmitted to the next point along the bar where bond exists until the section at this point is so reduced as to destroy the bond at that point.

While the data here presented are too meager to allow any definite conclusions to be drawn, the theory which suggests itself is that the total load required to cause slipping between a reinforcement bar and the concrete which embeds it when both the concrete and the steel are in tension depends upon:

1. The force with which the concrete grips the bar, which is a function of the character of the concrete and the surface condition of the bar;

2. The size of the bar;

3. The modulus of elasticity of the bar;

4. Poisson's ratio for the bar. It is obvious that from a practical standpoint this study is of little interest to the structural designer since this theory would have little effect upon the present standards of design and since the elongation of the bar necessary to destroy the bond throughout the length of a bar of any great length would cause failure of the structure.

However, it would seem to be of interest to highway engineers who have felt some concern about the possibility of the reinforcement bars in concrete pavement failing in tension at contraction cracks. This theory would show that the probability of such a failure is remote.

Its interest from a scientific standpoint is sufficient to cause us to pursue the study to determine if the accuracy of the theory can be demonstrated. Mr. Abrams' work will be of great value to us in this further study.

Mr. Slater.

MR. W. A. SLATER.[1]—The last conclusion in the paper, "The pull-out test of the form used in this investigation is a satisfactory form of specimen for comparative studies of bond," it seems to me,

[1] Engineer-Physicist, U. S. Bureau of Standards.

Mr. Slater.

is a little too positive. The thing that is needed in a bond test, I think, is a criterion of the bond resistance in a beam, since it is very seldom that we are concerned with bond except in a beam. If the beam is properly designed for a bond test an increase in bond resistance will cause a corresponding increase in maximum load. Therefore, with such a beam properly tested, the maximum load may be used as the criterion for bond resistance. Such a specimen has the advantage over a pull-out specimen that it removes the uncertainty as to whether the bond resistance at first slip, at maximum load or at an intermediate load should be used for comparison.

Recently I have been going over some test that were made in Germany a good many years ago[1] which, so far as I know, have never before gotten into the engineering literature of this country. They showed a decided advantage for the deformed bar over a plain bar when tested in a beam and I believe unpublished results of long-time tests of beams show a considerable advantage for the deformed bar. On the other hand the pull-out test shows that the first slip comes at about the same time for the deformed bar as for the plain bar. In the report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete[2] there is a statement that a deformed bar is to be considered as one which gives a bond resistance 25 per cent greater than that of a plain bar, without specifying the form of the test specimen. The form of the test specimen, therefore, is still an open question and I do not believe the data of this paper warrant any conclusion as to what is the correct form of test specimen for determining the relative merits of different types of bars.

I should like also to say something about Conclusion 7 regarding the effect of hydrated lime on bond resistance. It seems to me from an inspection of Fig. 11 that the generalization given in the conclusion is hardly warranted. It is true, of course, that there is a general decrease in bond resistance with the increase in the percentage of hydrated lime, but for two cases out of four, that is, at one year and at 7 days, there was an actual increase in bond resistance up to approximately 10 per cent of lime and a considerable increase at 5 per cent. The other two ages, 28 days and 3 months, although they showed somewhat of a decrease, decreased less rapidly with the small percentages than they did with the larger percentages of lime. It seems to me that it might be well to modify that conclusion, at least to the extent of not leaving the impression that the 1.2 per cent loss of bond resistance applies to each 1 per cent of hydrated lime. It is

[1] "Tests of Bond Resistance in Reinforced Concrete Beams," Abstract by W. A. Slater, *Engineering News-Record*, June 25, 1925 (Vol. 94, p. 1050).

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part I, p. 303 (1924).

Mr. Slater. very seldom that we want to use more than 10 per cent, and probably frequently not over 5 per cent in concrete, and with a quantity of that kind, whether it is a replacement or addition would matter less than with larger quantities.

Another feature that has some bearing on that question, I believe, is the fact that one of these groups (3 months tests) which I mentioned as showing a decreased bond resistance even with only 5 per cent of lime shows for that same percentage of lime a decided increase in the compressive strength; so that with at least two cases showing an increase, with two cases showing a decrease and with one of those which showed the decrease having some doubt thrown on its reliability by the fact that the compressive strength showed an increase, the conclusion is hardly justified. I think also consistency, or some more absolute measure of the workability of the different mixes, ought to be given, rather than simply the statement that it is a relative consistency of one.

Mr. Hutchinson. MR. G. W. HUTCHINSON.[1]—Most of us read only the conclusions of papers presented. I am not familiar with the practice of substitution of lime for cement and certainly do not believe in it. Cement, in normal mixtures, produces at least fifty per cent of the total strength of the mass while hydrated lime functions in making the mass more plastic.

By referring to the data in the paper itself, Mr. Smith's point seems to be well taken. With reference to conclusion No. 7, which refers to Tables V and VI, it does not appear to be consistent with the data presented. This is especially true when considered in the light of practical application of lime in the small percentages, rather than in mixtures not to be considered in other than theoretical studies.

I believe the claim is made that an addition of 1 per cent in the cement content produces from 1 to 1.5 per cent increase in the strength of concrete. Considering the effect of subtraction of cement proportional to that of addition, an average decrease of 1.25 per cent strength should be secured for each 1 per cent of cement taken away for lime substitution.

It is generally conceded that tests at the later period are more dependable than those made at the earlier periods. In tests containing the amount of hydrated lime generally recommended for additions, say 10 to 20 per cent by volume, we should find, according to conclusion No. 7, that the strength of the concrete is reduced approximately 12 to 24 per cent. At the age of one year the decrease in strength according to Table V is but 0.5 per cent instead of 12 per

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cent with the 10-per-cent substitution, and but 5.5 per cent instead of 24 per cent, in the case of the 20-per-cent substitution. The bond strength appears to be affected similarly. Mr. Hutchinson.

There must be some reason for this great difference between the results such as given in Table V, and those secured by subtracting cement when lime is not used. I believe it is generally recognized that hydrated lime increases the workability of concrete. Workability is one of the most important factors to be considered in field concrete. I am wondering if any other but this property, and its accompanying advantages, imparted in this case by the use of lime, could be responsible for overcoming the great difference between the well-recognized effect of cement on the strength of concrete and the effect as given in Table V if interpreted on a practical basis. An actual decrease in strength would be obtained at such point as the advantages gained by increased workability are overbalanced by the decrease in volume of the cementing ingredient. This would vary with difference mixtures.

MR. DUFF A. ABRAMS.—This paper was not intended to be a paper on hydrated lime in concrete. That is a big question which has been before the Society previously. Data of tests were presented in which hydrated lime was both added to and replaced cement.[1] The fact that the hydrated lime in these tests replaced cement should not be given too much significance; it is a perfectly straightforward matter to convert one set of mixtures into terms of the other. Mr. Abrams.

When a powdered material is put in concrete, we are not only disturbing the relation between the quantities of cementing material, but we are changing the nature of the aggregate and the state of workability of the concrete and what is of more importance, we are disturbing the relation between the water content and the cement. All of these factors influence the results in such a way that it is very difficult to determine what actually happens.

We do not expect, in tests of this kind, that each point on the diagram will fall in exactly the place where a smooth curve can be drawn through it. Any point is subject to discrepancies and a number of such discrepancies are shown in Fig. 11. The curves were drawn in an effort to indicate the trend of values; it may be expected that certain points will fall above and others below the curve.

It has been pointed out that a number of the curves show slight increases in strength for the smaller percentages of lime. That claim might be made; on the other hand the general trend unquestionably indicates a reduction in strength.

[1] *Proceedings*, Am. Soc. Testing Mats., Vol. XX, Part II, p. 149 (1920); Vol. 22, Part I, p. 292 (1922).

Mr. Abrams. Three of the points for 5-per-cent lime show an increase in strength or bond as compared with no lime; one shows no change; three show a proportional reduction; and one shows a decided reduction. Any of the values are subject to an error of say 5 to 10 per cent. If we consider that three of the points without lime (7 days and 1 year bond and 3 months compression) may be low due to accidental variation by as much as 5 per cent there would then be little reason to consider any increase due to lime. The diagram shows numerous instances where the values are obviously out of line; for example, 5 per cent lime in compression at 28 days actually gave 2410 lb. per sq. in., whereas about 2700 would have been necessary to bring this point up to the curve as drawn, in other words this point is in error about 12 per cent.

The conclusion stated with reference to hydrated lime is fully borne out by the more comprehensive studies referred to above.

Mr. Slater refers to the methods of making bond tests and the relation of pull-out tests to the bond developed in beams. That subject was covered by a comprehensive series of tests made by the speaker at the University of Illinois.[1] Similar tests were later carried out under Mr. Slater's direction for the U. S. Shipping Board.[2]

Tests made by the speaker at the University of Illinois on simple beams reinforced with relatively large bars and loaded at the third points showed that bond stress was not distributed uniformly as our theory of reinforced concrete beam action implies, but that the maximum bond stress was developed just outside the load points and comparatively early in the loading. The ultimate bond resistance was then developed progressively toward the ends of the bar. First end slip of bar was coincident with failure of the beam in diagonal tension, whether plain or deformed bars were used.

The University of Illinois tests showed that the true bond resistance was measured by the pull-out test. Two different methods gave a measure of the bond resistance over an infinitesimal area:

1. Pull-out tests of bars of different diameters using a constant embedment furnished an estimate of the bond resistance of a bar of infinitesimal diameter.
2. Pull-out tests of bars of a constant diameter with different embedments gave the bond resistance of a bar having infinitesimal embedment.

The results of tests in 1 and 2 were the same.

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[1] "Tests of Bond Between Concrete and Steel," by Duff A. Abrams, *Bulletin 71*, Eng. Experiment Station, University of Illinois, Urbana, Ill.

[2] "Tests of Bond Resistance Between Concrete and Steel," by W. A. Slater, F. E. Richart and G. G. Schofield, *Technologic Paper No. 173*, U. S. Bureau of Standards.

Unless the bar is anchored in a column or abutment there is not a great deal of difference in the actual bond resistance developed by plain and deformed bars. Deformed bars must slip an appreciable amount before the deformations come into action. Mr. Abrams.

Reference was made to the Joint Committee definition of a deformed bar as being one which must develop 25 per cent greater strength than a plain bar of the same size and so on. The members probably recognized the limitations of this definition in that it did not specify a method of test. However, it does indicate the direction in which the committee was aiming, that is the type of deformed bar that would give an increased resistance, possibly as determined by a pull-out test. A deformed bar which would give 25 per cent greater bond strength in a pull-out test without splitting the test block probably would have as much merit as any deformed bar, although its merits would be from the standpoint of *anchorage* and not from any appreciable increase in true bond resistance in a beam.

The data submitted by Messrs. Crum and Myers show that the load at first slip was at or near the yield point of the $\frac{1}{2}$-in. round bars; consequently they are in error in concluding that the total load necessary to cause slipping between plain round steel bars and concrete is independent of the length of the bar or the area of contact. When the yield point of the steel is reached there is, of course, a marked reduction in cross-sectional area which immediately destroys bond and greatly modifies the test conditions. These tests merely show that the length of embedment was sufficient to develop the yield point of the steel before end slip occurred. After the yield point of the steel was reached, as pointed out by Crum and Myers, there was a progressive failure in bond along the length of the bar which culminated in slip of the bar at end of specimen at a total load somewhere between the yield point and the ultimate strength of the steel. Crum and Myers properly point out that this condition would not arise in a typical reinforced concrete member. This type of test does not measure bond in any sense of the term and gives no information that could not have been arrived at from a consideration of the elementary principles of mechanics.

After having made many hundreds of pull-out tests of all types of concrete and many different forms of bars, and having considered the criticisms of this test which have been presented, the speaker is still of the opinion that it is an excellent form of specimen for studying bond between concrete and steel. The specimen must be properly designed to avoid over stressing either the steel or the concrete; this means that the embedment must not be too great or the bar too small.

Mr. Abrams. The steel should not be stressed to more than say 50 per cent of its yield point, the concrete to not more than 20 per cent of its ultimate compressive strength. These conditions are met by using a 1-in. bar embedded 8 in.

To use reinforced concrete beams in a study of bond is to substitute a complex, laborious and uncertain method for a very simple and direct one. Mr. Slater seems to have reversed the favorable opinion of the pull-out test which is expressed on page 9 of *Technologic Paper No. 173*, referred to above.

ANALYTICAL PROPERTIES OF SET AND HARDENED MORTARS[1]

By E. E. Butterfield[2]

Synopsis

This paper describes tests by which the cement content may be calculated from the determination of calcium oxide in hardened mortars proportioned by the method of mortar voids or with inundated sand with an average variation of less than 1 per cent.

In mortars separated from masses of concrete of 1.5 cu. ft. and upwards, the cement content determined by analysis is lower than the percentage of cement used by weight of dry materials. This is due in part to the hydration and carbonation of cement in the mortar, partly to the loss of cement in mortar adherent to the forms and probably in part to the percolation of mortar into a porous subgrade. In four specimens of concrete containing limestone coarse aggregate the average difference between cement used and cement found was −5.7 per cent and the maximum difference was −15 per cent. Generally the cement content of two different samples of mortar from the same mass of concrete will vary more than the difference between the cement used and the average value of the cement content of two samples from the different masses of concrete of the same nominal proportions.

In continuation of a study of some of the analytical properties of mortars and concrete a series of mortars proportioned with inundated sand and a series of mortars separated from mass concrete of known porportions have been subjected to the methods of analysis described in a paper presented last year by the author.[3]

Mortars Proportioned with Inundated Sand

According to A. N. Talbot's method for the determination of mortar voids,[4] dry sand and cement are mixed with water and remixed with small increments of water until the point of minimum voids is reached and passed. In last year's series of twenty mortars about 10 per cent of water in excess of the quantity at the point of minimum voids was added to produce a plastic mass which was used for the molding of the specimens for analysis.

[1] From the Laboratories of the President of the Borough of Queens, New York City.

[2] In Charge of Laboratory, Borough of Queens, New York City.

[3] E. E. Butterfield, "Methods of Securing Samples of Completed Pavements, with Reference to the Determination of the Quality of the Cement-Concrete Foundation," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 1066 (1924).

[4] A. N. Talbot, "A Proposed Method of Estimating the Density and Strength of Concrete and of Proportioning the Materials by the Experimental and Analytical Consideration of the Voids in Mortar and Concrete," *Proceedings*, Am. Soc. Testing Mats., Vol. 21, p. 940 (1921).

In proportioning mortars of the present series with wet sand, a measured and weighed quantity of dry sand was poured into a graduated cylinder containing a volume of water more than sufficient to thoroughly wet the sand and the resulting volume was noted. The wet sand was then thoroughly mixed with cement and the mortar tamped in the brass mold used for the determination of mortar voids. After the weight of a known volume of mortar had been obtained,

TABLE I.—PROPERTIES OF FRESH MORTAR.

Sand measured in water.
Ratio of weight of dry sand to volume of thoroughly wet sand varies from 0.592 to 0.604.
Percentage of water by weight of dry sand varies from 24 to 26.
Specific gravity of cement, 3.15.
Specific gravity of sand, 2.67.
Volume of mold, 361 cc.

	Ratio of Absolute Volumes of Sand and Cement, $\frac{a}{c}$	Weight Ratio	Bulk Ratio, Wet Sand	Weight of Cement, g.	Weight of Dry Sand, g.	Volume of Wet Sand, cc.	Volume of Water, cc.	Weight of Mortar in Mold, g.	Volume of Batch, cc.	Mortar Voids v_m
No. 1	2.5	1:2.1	1:1.9	215	456	270	112	767	368	0.35
No. 2	3.8	1:3.2	1:2.9	165	535	323	137	752	402	0.37
No. 3	5.9	1:5.0	1:4.5	125	628	373	150	737	442	0.37
No. 4	6.5	1:5.5	1:5.0	115	631	378	161	741	442	0.38
No. 5	7.8	1:6.6	1:6.0	100	658	393	159	733	452	0.38

TABLE II.—PROPERTIES OF HARDENED MORTAR. MORTAR FROM TABLE I EXAMINED IN AIR-DRY CONDITION.

CaO in cement, 62.0 per cent. CaO in sand, 0.17 per cent. Soluble silicates in cement, 21.2 per cent.

	Age of Mortar, days	Volume of Specimen, cc.	Weight, g.	Apparent Specific Gravity of Whole Specimen	Apparent Specific Gravity of Mortar Fragments	Specific Gravity of Ground Mortar Passing 80-mesh	Cement Calculated from Weights of Mortar and Cement, per cent by weight	CaO, per cent by weight	Soluble Silicates, per cent by weight
No. 1	42	339.7	685.5	2.018	2.013	2.593	31.3	18.78	6.03
No. 2	42	369.7	713.7	1.930	1.925	2.595	23.1	13.97	4.52
No. 3	43	408.4	761.0	1.863	1.869	2.625	16.4	9.86	3.39
No. 4	42	402.5	757.2	1.881	1.882	2.625	15.2	9.14	3.12
No. 5	43	418.9	767.6	1.832	1.857	2.622	13.0	7.92	2.85

the whole batch was remixed and tamped in a cylindrical mold with a solid bottom and allowed to set and harden.

The properties of the mortar in the fresh and plastic condition are given in Table I.

Mortar Voids.—The voids in the inundated sand mortars vary from 0.35 to 0.38. In last year's series of twenty mortars proportioned by Talbot's method with dry sand and just sufficient water to produce a workable mortar, the mortar voids varied only between 0.32 and 0.33. The voids in mortars proportioned with inundated sand are therefore greater and more variable than in mortars proportioned by

Talbot's method. Although there is a greater constancy between weight and volume of sand, due to the method of measuring uniformly wet sand, this does not tend to produce more uniform mortar. The voids in both series of mortars are represented graphically in Fig. 1.

The properties of the hardened mortar examined in air-dry conditions are given in Table II.

Apparent Specific Gravity of Mortar.—The determinations of the apparent specific gravity of mortar fragments by displacement in water of paraffin-coated fragments weighing 7 to 32 g. agree well

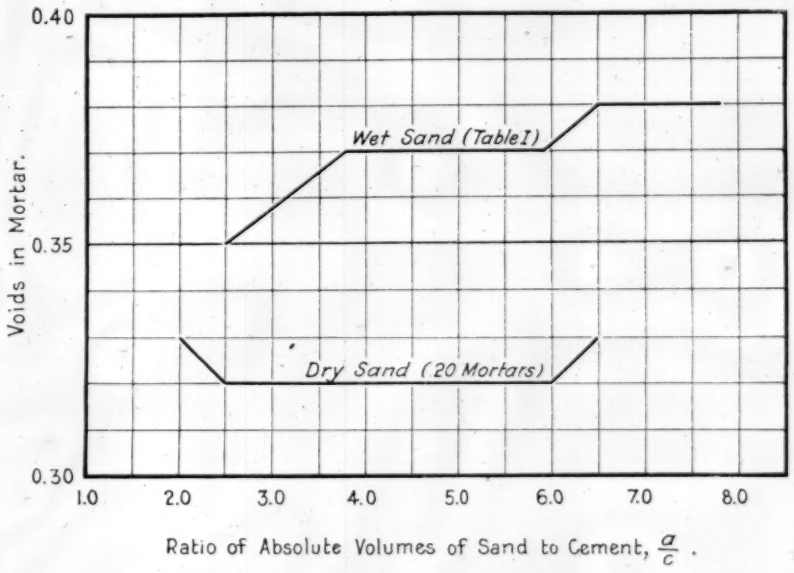

FIG. 1.—Comparison of Voids in Mortars Made With Dry Sands and Inundated Sands.

with the apparent specific gravities of the whole specimens calculated from weight and volume, so that the paraffin method appears to be a reliable method for the determination of the apparent specific gravity of hardened mortar. The apparent specific gravity of the leaner mortars from $a/c = 3.8$ to $a/c = 7.8$ is appreciably lower than the apparent specific gravity of corresponding mortars proportioned by Talbot's method, due to the greater voids in the mortars proportioned with wet sand.

The Specific Gravity of Ground Mortar passing 80-Mesh is of about the same order as in mortars proportioned by Talbot's method for mortar voids. The specific gravity of the powdered mortar is

not equal to the sum of the products of the fractional parts of original materials used multiplied by the respective specific gravities, but it is appreciably lower. This is due to the hydration and carbonation of the cement, resulting in the formation of products with lower specific gravities than the original constituents of dry portland cement.

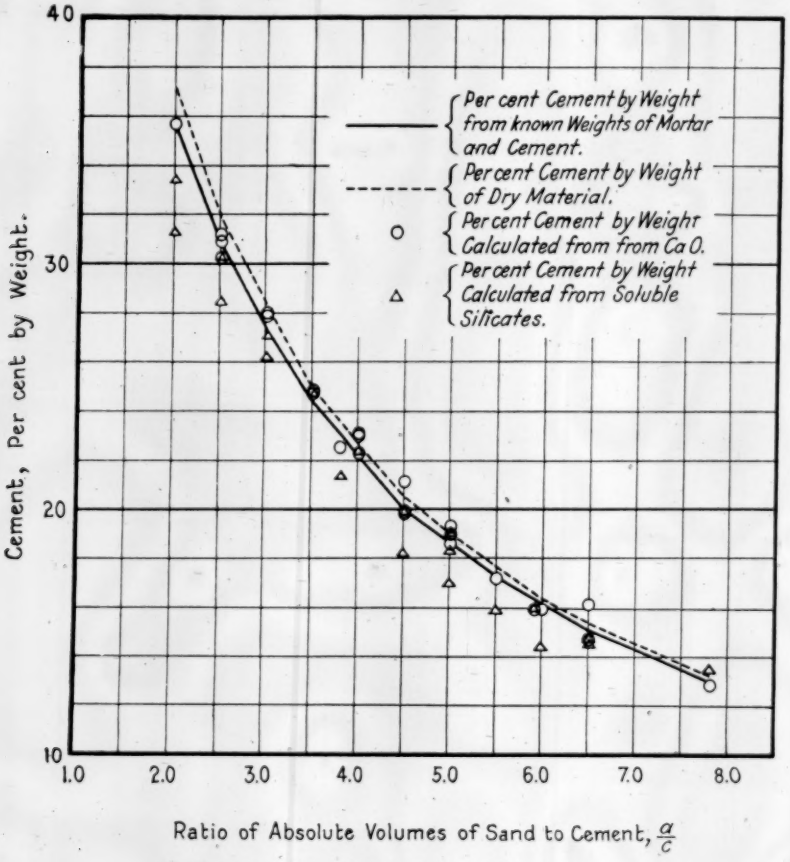

FIG. 2.—Chemical Analysis of Mortars.

From the data on the specific gravity of the mortar in conjunction with the weights of materials used it is possible to arrive at some figures for the specific gravity of "hydrated" cement. From the data on twenty-five mortars, including the mortars in Table II, the specific gravity of "hydrated" cement appears to vary from 2.34 to 2.52, the average of all being 2.43. Specific gravities of 2.42 and 2.47 were found on neat cement cubes which had been stored in water for 28 days and in air from 7 to 21 days, dried at 105° C. to constant

weight and ground to pass 80-mesh. It is fully realized that these figures for the specific gravity of "hydrated" cement do not represent a constant for a substance of uniform chemical composition, but they represent rather the mean specific gravity of a mixture of different substances resulting from the hydration and carbonation of some of the components of portland cement together with water which is not expelled at 105° C. The figures also have a bearing on the condition that the percentage of cement by weight of hardened mortar is always slightly lower than the percentage of cement by weight of the original dry materials, since the weight of the hardened mortar has increased over the weight of the dry materials according to the amount of hydration and carbonation of the cement in the mortar.

Cement Content.—For the analytical determination of cement two methods were used: (1) the gravimetric determination of calcium oxide and (2) the determination of soluble silicates according to the method of H. F. Kriege.[1] The actual cement content was figured both from the percentage by weight of dry materials and the percentage by weight of hardened mortar. The results on the present series together with the results of twenty mortars of last year's series are summarized in Fig. 2. The percentage of cement by weight of dry materials and the percentage of cement by weight of air-dry mortar are almost parallel throughout the upper half of the curves, tending to converge in the leaner mixes due to the diminishing quantities of products of hydration and carbonation of the cement as the cement itself becomes less in quantity. The maximum variations in the cement calculated from the calcium oxide content are from −3.5 to +7.3 per cent of the actual percentage of cement by weight of hardened mortar. The agreement is generally good and the average deviation is +0.75 per cent. The maximum variations in the cement calculated from the soluble silicates are from −11.4 to +3.0 per cent of the actual percentage of cement by weight of hardened mortar. The variations are greater and the results tend to run lower than either the cement content of the mortar or the results from the calcium oxide determinations. The average deviation is −4.3 per cent.

In considering the results it should be borne in mind that all mortars were made with sand containing a negligible amount of calcium, that the whole batch of each mortar was molded in a metal vessel open only at one end and that the mortars were cured in laboratory air. Therefore, the washing away of cement with excess mixing water and the leaching out of free lime by the percolation of

[1] H. F. Kriege, "Determining the Cement Content of Concrete," *Engineering News-Record*, Vol. 92, p. 892 (1924).

water through the mortar has not occurred as it would in the case of mortars stored in water or in the case of concrete mixed on the job and exposed to rain or abnormal conditions of subgrade moisture.

MORTAR SEPARATED FROM MASS CONCRETE

It is well to bear in mind at the start that mass concrete is far from being a uniform product. Take, for example, one of the simplest properties of concrete, the weight per unit volume. On sixteen slabs 1½ in. thick cut from a block of 1:3:6 concrete 30 by 13 by 6 in. the weight varied from 143.4 to 152.5 lb. per cu. ft. On twenty-five 4-in. cubes cut from 1:3:6 pavement base the weight varied from 148.3 to 157.6 lb. per cu. ft. Therefore it is reasonable to expect that the variations in the cement content of the mortars separated from cast blocks of concrete 1.5 cu. ft. and upwards will be greater than in mortar cylinders carefully proportioned and mixed in the laboratory and weighing only 1 to 2 lb. each.

The concrete from which the mortar was obtained consisted of blocks of about 2 cu. ft. of 1:3:6 concrete from which 4-in. cubes were cut for compression tests in connection with a series of tests for Sub-Committee III of Committee C-9 on Concrete and Concrete Aggregates. In order to approximate natural conditions, an artificial subgrade consisting of 4 in. of rammed sand-clay top soil was placed on the concrete floor of the laboratory. The artificial subgrade was kept moist and freshly wetted immediately before depositing any concrete. Wooden forms were placed on the artificial subgrade. The concrete was mixed by hand in one-quarter bag batches and all materials were weighed as well as measured by loose volume. Two consistencies were used, one with just sufficient water to be freely workable and the other decidedly sloppy. The concrete was thoroughly mixed, deposited in the forms, and spaded. After 3 days, the forms were pulled and the concrete was covered with damp sand which was sprinkled with water from day to day. At the end of 3 weeks the blocks were sawed into 4-in. cubes for the compression test at 28 days. After the compression test the mortar was hand picked from the broken cubes. Two sets of broken cubes weighing 22.5 lb. each were selected from each original batch. From each 22.5 lb. of concrete ½ to 1 lb. of picked mortar was obtained. As there were two separate analyses on each batch the total quantity of concrete taken for analysis from each batch was 45 lb. which yielded about 1½ lb. of mortar suitable for analysis. The mortar was ground to pass 80-mesh and subjected to the same methods used for the analysis of the mortar voids series and the inundated sand series. The results are given in Table III.

Variability of the Mortar from the Same Mass of Concrete.—The variation in the cement content from point to point in the mortar in the same mass of concrete, as judged by the analysis of two separate samples of mortar (designated *a* and *b* in Table III), is greater in all cases, except concrete No. 5, than is the difference between the cement content calculated from the weights of dry materials and the average value of the two analyses on the mortar from the different batches of concrete of the same nominal proportions. Practically, with reference to the size of the samples, this means that a single 22.5-lb. sample from 270 lb. of concrete is not sufficient for an approximate determination of the cement content of the mortar in the 270-lb. mass. Not until we have the average of two separate analyses representing 45 lb. of concrete and about $1\frac{1}{2}$ lb. of clean mortar, do we

TABLE III.—RESULTS OF ANALYSIS OF MORTAR FROM 1:3:6 CONCRETE.

Cow Bay sand as fine aggregate.

	Coarse Aggregate	Slump Test, Fracional Drop of 6 by 12-in. and 8 by 16-in. Cylinders	Ratio of Sand to Cement by Weight	CaO in Mortar, per cent by weight		Cement in Mortar by Weight of Dry Materials, per cent	Cement Calculated from CaO, per cent by weight		
				Sample *a*	Sample *b*		Sample *a*	Sample *b*	Average
No. 1	$\frac{3}{4}$-in. Silicate Gravel	$\frac{1}{3}$	1:3.5	11.9	12.9	22.3	19.2	20.8	20.0
No. 2	$\frac{3}{4}$-in. Silicate Gravel	$\frac{2}{3}$	1:3.5	13.3	11.1	22.3	21.4	17.9	19.7
No. 3	$\frac{3}{4}$-in. Limestone.....	$\frac{1}{3}$	1:3.1	13.5	16.1	24.4	21.7	25.8	23.8
No. 4	$\frac{3}{4}$-in. Limestone.....	$\frac{2}{3}$	1:3.1	12.9	14.4	24.4	20.8	23.2	22.0
No. 5	$1\frac{1}{2}$-in. Limestone...	$\frac{3}{8}$	1:3.2	13.2	14.2	24.0	21.3	22.8	22.1
No. 6	$1\frac{1}{2}$-in. Limestone...	$\frac{3}{4}$	1:3.1	16.0	13.0	24.1	25.8	20.9	23.4

get anywhere near the original cement content with any degree of regularity. In the most extreme case, concrete No. 4, the analysis of sample *a* alone would lead to the appraisal of the mortar which is actually 1:3.1 by weight as a 1:3.8 mortar by weight, indicating a shortage of 15 per cent of cement.

Silicate Gravel Concrete.—The purpose of the silicate gravel series was to determine the difference in the calcium oxide content of mortars from concrete containing no limestone as compared with mortars contaminated with limestone dust and small particles of limestone from the coarse aggregate. As a matter of fact, our mortars made with silicate gravel and silicate sand are really unsuitable for analysis or appraisal of the properties of the mortar alone, on account of the overlapping of sizes of particles of the same mineralogical nature in the sand and gravel. The gravel used contained 19.2 per cent of

material passing a $\frac{3}{8}$-in. sieve and retained on a $\frac{1}{4}$-in. sieve and 6.4 per cent of material passing a $\frac{1}{4}$-in. sieve. Roughly 25 per cent of the coarse aggregate was close to the size of sand particles and 6 per cent was actually of the size of particles of coarse sand. This condition creates difficulties in a proper calculation of the ratio of cement to sand and consequently in the estimation of the cement content of the mortar. While the proportion of cement to commercial sand was 1:3.1 by weight, the actual proportion in the concrete of cement to material passing a $\frac{1}{4}$-in. sieve was 1:3.5. When it comes to picking out mortar fragments from the crushed concrete it is difficult to draw the line at fragments about $\frac{1}{4}$-in. as to whether they are to be regarded as mortar or as particles of coarse aggregate. As a consequence of this indefiniteness, no doubt, the cement content derived from chemical analysis of the mortar is 9 per cent less than the cement content calculated from the weights of materials under the assumption that all of the material in the concrete passing a $\frac{1}{4}$-in. sieve is a constituent of the mortar. To make a proper comparison of mortars of the same proportions from concrete containing silicate gravel coarse aggregate and concrete containing limestone as coarse aggregate, it will be necessary to use one size gravel above $\frac{3}{8}$ in. in size so that there will be a sharp line of demarcation between coarse aggregate and mortar.

Limestone Concrete.—The limestone used contained no material passing a $\frac{1}{4}$-in. sieve and the sand contained practically no calcium compounds. The separation of mortar and coarse aggregate is clean cut and as the results of analysis of the mortar are lower than the quantity of cement used in the mortar, the contamination of the mortar with limestone dust appears to be negligible or if it does occur it is offset by other factors. It will be noted that there are differences of -0.6, -2.4, -1.9 and -0.7 per cent respectively between the cement used and the average cement content found by analysis. Part of this difference is due to the difference between the cement by weight of dry materials and the cement by weight of hardened mortar. In Fig. 2 it will be seen that 24.0 to 24.4 per cent of cement by weight of dry materials corresponds to 0.5 per cent less of cement by weight of hardened mortar. As we can only determine by analysis the percentage of cement by weight of hardened mortar it is necessary to add 0.5 per cent to the figure found by analysis to make a proper comparison with the percentage by weight of dry materials used. After this correction is made there are appreciable differences in concretes Nos. 4 and 5 of -1.9 and -1.4 per cent of cement, respectively, to account for. On removing the forms and

hoisting the concrete slabs from the artificial subgrade, it is the usual thing to find particles of rich mortar in the corners and on the edges of the forms and also to find about 2 in. of top soil adhering to the bottom of the concrete. Just how much cement is lost by percolation into the subgrade and by the spalling off of fragments of rich mortar on the forms will be determined in further tests.

From the present results it is apparent that bona fide 1:3:6 concrete may show on uncorrected analysis a cement to sand ratio as high as 1:3.8 and an average of 1:3.4. The results should be taken into consideration in attempts at the approximation of the cement content of job concrete of nominal 1:3:6 proportions.

A series of six slabs of concrete may seem too small to warrant general conclusions, but we have further data in the course of preparation on twenty batches of concrete up to two-bag mixes which seem to be in accord with the present results.

DISCUSSION

Mr. Richart.

MR. F. E. RICHART[1] (*presented in written form*).—The methods of studying the constitution of samples of hardened mortar and concrete given by Mr. Butterfield in his paper and in the one presented last year[2] appear to be very useful. Increased attention is being given to the quality of concrete in service but for most concrete on which a "*post mortem*" examination is to be made there is little information on the ingredients or details of its placing. Simple methods for analyzing a sample of concrete to determine the proportions of its constituents will be welcomed by engineers.

The correlation of the proportions of the ingredients of the hardened concrete with the proportions of the original dry materials is especially desirable. The density of concrete is commonly defined as the proportion of solids in the freshly placed concrete; a record of this property is rarely available for concrete existing in structures, and little is known regarding the relative densities of freshly mixed and hardened concrete. As Mr. Butterfield has shown, there is a slight decrease in the volume of cement mortar from the time of placing until it has taken its initial set, a decrease that is greatest for wet and rich mixes. Beyond this there is a further slight shrinkage which continues indefinitely. These external reductions in volume are accompanied by the formation of hydration products of the cement which take up more space than the original cement particles. Hence, a determination of the voids in hardened mortar by a comparison of the apparent specific gravities of a specimen in the unbroken and in the finely powdered conditions, or by a total absorption test similar to that used to determine the porosity of brick, will necessarily indicate much smaller voids than those to be found in the freshly placed mortar. From a study of the data of Mr. Butterfield's tests it appears that for the particular mixtures used, the density of the hardened mortar was from 1.09 to 1.16 times that of freshly placed mortar, with the ratio nearest unity in the leaner mixtures where the hydration products of the cement were least in importance.

[1] Research Assistant Professor, Engineering Experiment Station, Laboratory of Applied Mechanics, University of Illinois, Urbana, Ill.

[2] E. E. Butterfield, "Methods of Securing Samples of Completed Pavements with Reference to the Determination of the Quality of the Cement-Concrete Foundation," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 1066 (1924).

Mr. Richart.

Referring to Mr. Butterfield's observations as to the voids in mortars made with dry and inundated sands, it appears that the results shown in his Fig. 1 are not sufficiently explained and may be misunderstood. If equal amounts of mixing water were used in the two cases, there is no apparent reason why inundation of the sand before mixing it with the cement should give voids appreciably different from those obtained by mixing sand and cement with water in the usual way. If there was a difference due to a greater absorption of water by the sand when inundated it would tend to decrease rather than to

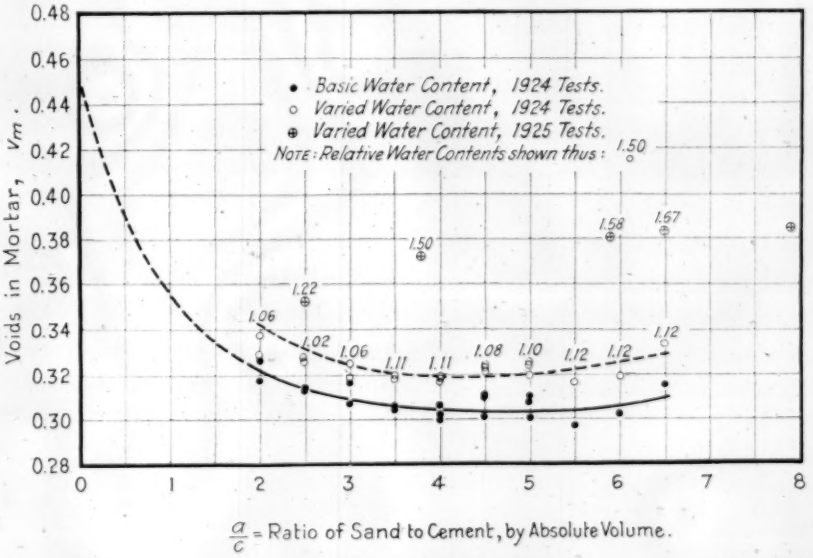

FIG. 1.—Relation of Voids in Mortar to Ratio of Sand to Cement for Mortar of Various Water Content.

increase the voids in the mortar made with inundated sand, assuming that equal allowances for absorption were made. Apparently the conclusion to be emphasized regarding the tests with inundated sand is that lean mortars of 1.10 relative water content cannot be made using this process, since the water required to inundate the sand is much more than that needed as mixing water in the mortar. A study of the tabulated data of Mr. Butterfield's tests shows that in the mortars made with inundated sand the water content was 20 to 60 per cent greater than for the corresponding mortars made last year with dry sand, assuming that identical lots of sand and cement were used; this increase in water used in the 1925 tests was more than enough to produce the increase in mortar voids attributed to the

Mr. Richart. innundation method. As shown in the paper by Talbot, to which reference has been made, an increase in water content produces a corresponding but somewhat smaller increase in mortar voids (since air voids are being reduced at the same time); an increase from 1.00 to 1.40 relative water content causes an increase in mortar voids of 10 to 35 per cent of their value, with different sands and richness of mix.

The mortar-voids curves of Fig. 1 of the paper might be expected to take a typical curved form rather than the combinations of straight lines shown. The lower curve, for which the mortar had roughly a 1.10 relative water content, might be plotted as shown in the accom-

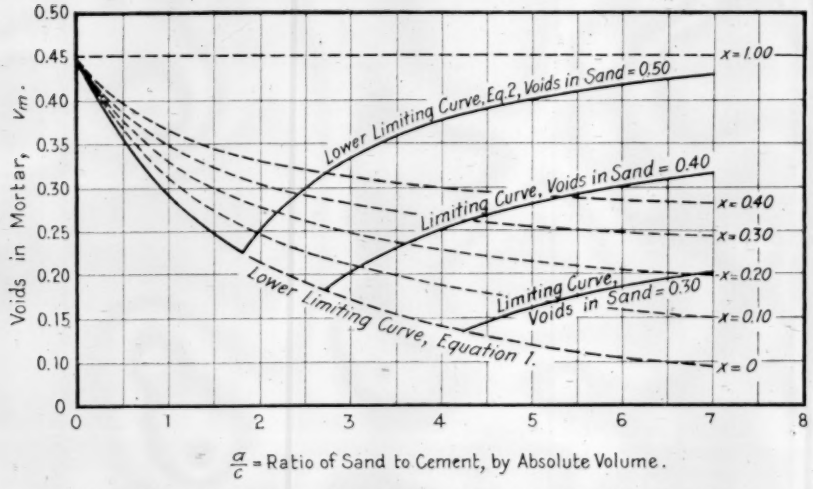

FIG. 2.—Relation of Voids in Mortar to Ratio of Sand to Cement.

panying Fig. 1. The upper curve means little since the water content varied greatly. There are certain theoretical considerations which quite definitely limit the shape of the mortar-voids curve and these may be considered briefly.

A neat cement mortar at basic water content (minimum voids) has voids of about 0.45. Now if a few sand grains were mixed into a mass of neat cement paste to replace an equal volume of paste, obviously the voids would be reduced. Let v_m denote the proportion of voids in freshly placed mortar of a given water content, v_n the proportion of voids in neat cement mortar of the same relative water content, a the absolute volume of sand in the batch and c the absolute volume of cement in the batch; then for a mortar in which there is

sufficient paste to surround completely the sand particles, the following equation may be derived: Mr. Richart.

$$v_m = \frac{v}{1+\frac{a}{c}(1-v_n)} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

As noted before, at basic water content $v_n = 0.45$, Eq. 1 is represented by the lower curve of the accompanying Fig. 2, which forms a lower limiting envelope of all basic mortar-voids curves that may be drawn for different sands.

For the case in which there is insufficient cement paste to surround the sand particles, consider first the sand alone as a mortar with an infinite value of $\frac{a}{c}$. It is known that with the addition of water to sand it first bulks or swells, then with more water it reaches a condition of minimum voids, much as mortar does. Let the minimum voids in the sand at its basic water content be denoted by v_a. (These voids are generally slightly greater than the voids in dry sand obtained by the method of tamping given in A.S.T.M. Standard Method of Test for Unit Weight of Aggregate for Concrete, C 29 – 21.)[1] Now if cement particles are added to the sand to form a very lean mortar, the voids in the mortar will comprise the voids in the sand minus the absolute volume of the cement particles, assuming that the cement particles do not "wedge apart" the sand grains. The proportion of mortar voids may then be given by the equation:

$$v_m = v_a\left(1+\frac{c}{a}\right) - \frac{c}{a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Eq. 2 indicates that the voids in lean mortars depend upon the voids in the wet sand, which in turn depend upon its gradation. For a sand with a given value of v_a, the equation gives a second lower limiting envelope for mortar-voids curves, as shown in Fig. 2. It applies to mixtures in which the voids in the sand are not well filled with cement paste and which are likely to be non-homogeneous. The intersection of the curves representing Eqs. 1 and 2 gives the mixture at which the volume of cement paste is just equal to the voids in the sand. For this case, $v_m = v_a v_n$, and the proportions of the mixture are given by the equation $\frac{a}{c} = \frac{1-v_a}{v_a(1-v_n)}$. According to the foregoing analysis, this mixture should give the smallest possible mortar voids with the given sand and cement.

[1] 1924 Book of A.S.T.M. Standards.

Mr. Richart. The derivation of Eq. 1 has presupposed the use of large sand particles, around which the cement paste would fit closely just as mortar is known to fit around particles of coarse aggregate. If the sand particles were as fine as the cement, it seems likely that such

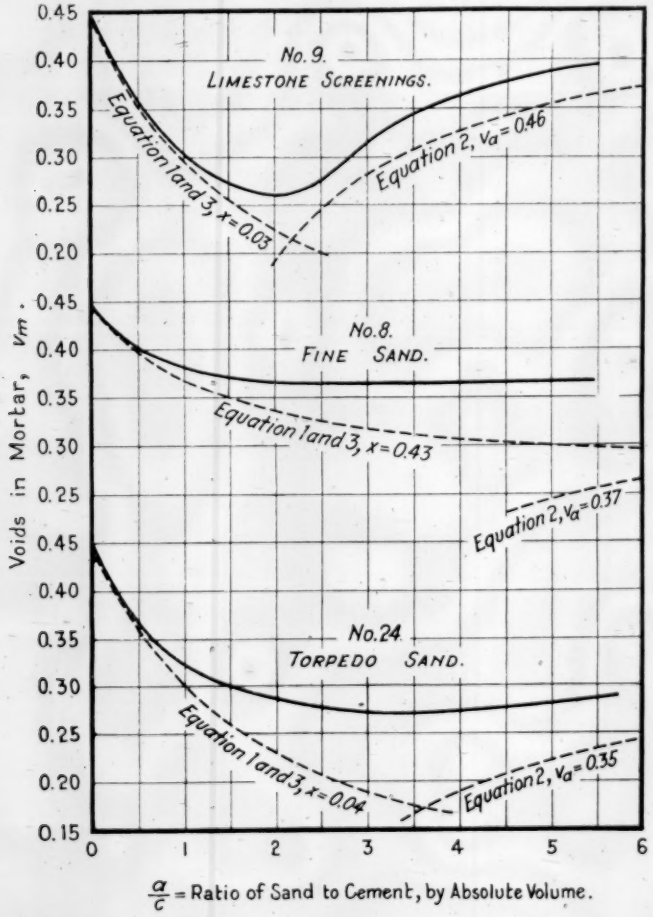

FIG. 3.—Relation of Voids in Mortar to Ratio of Sand to Cement for Mortars Containing Various Types of Sand.

close contact would not result and the mortar might be expected to have as great voids as the neat cement, regardless of the proportions of the mix. Actually, natural sands do contain a small amount of material as fine as cement and such material may be considered to act as cement rather than sand in its effect upon the mortar voids. Hence, the part of the sand passing the No. 100 sieve (including some material

slightly coarser than the cement) might be considered as cement in the application of Eq. 1. If the proportion of the sand passing the No. 100 sieve is denoted by x, the modified value of $\frac{a}{c}$ to be used in Eq. 1 becomes: **Mr. Richart.**

$$\frac{a'}{c'}=\frac{a}{c}\left(\frac{1-x}{1+\frac{a}{c}x}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

The effect of various proportions of 0-No. 100 sand upon the limiting mortar-voids curve is seen in Fig. 2, wherein curves for $x=0.10$, 0.20, 0.30 and 0.40 are shown. Evidently, a clean, coarse sand tends to follow the curve of Eq. 1, while a very fine sand will have a much higher limit for its mortar-voids curve.

A comparison of actual mortar-voids curves with the ideal curves of the preceding study shows a general agreement in shape, but the actual mortar voids in all cases greater than the analytical values, due to imperfect fitting together, or wedging action, of the particles. Evidently the mortar-voids curves, while governed largely by the analytical relations heretofore stated, also depend upon the individual characteristics of the sand such as the condition and character of the surface, absorptive properties, shape, size and gradation of particles as affecting wedging action and bulking. The accompanying Fig. 3 shows curves for mortar voids at basic water content, for fine aggregates of three distinct types.

Fine aggregate No. 9 is limestone screenings consisting mainly of particles between the No. 4 and No. 8 sieves. It had voids of 0.46 and had 3 per cent passing the No. 100 sieve. The curve shows the least voids where $\frac{a}{c}$ is about 2, with a rapid increase at either side of this point. This shape of curve is typical for coarse sands of uniform size, which have high voids and a small percentage of 0-No. 100 particles. Standard Ottawa testing sand is of this class.

Sand No. 8 is a very fine silica sand, having voids of 0.37 and containing 43 per cent passing the No. 100 sieve. It will be seen that the intersection of the limiting courves is beyond the limits of the figure, at a value of $\frac{a}{c}$ of 7.8. Sands of this type have quite uniformly high mortar voids throughout the range of proportions commonly used.

Sand No. 24 is a well-graded torpedo sand, with the greatest variance between the limiting curves and the actual experimental curve of the three examples shown. This sand, with voids of 0.35 and 4 per cent of 0-No. 100 particles, has the intersection of the limiting

Mr. Richart. curves at a value of $\frac{a}{c}$ of 3.6. The mortar voids are quite low over a considerable part of the curve, indicating that the sand will produce high strengths when used over a wide range of mortar mixtures.

While only a rough agreement is seen between the experimental curves and the limiting curves suggested by the analysis, the agreement is very close in wetter mixes wherein the particles are better lubricated, and it seems certain that the general shape of the curves is governed largely by the gradation of the sand as reflected in the sand voids and the proportion of very fine material.

Mr. Butterfield. MR. E. E. BUTTERFIELD.—I desire to thank Mr. Richart for his valuable discussion and if the only result from my paper were to call forth the discussion of Mr. Richart I would be satisfied with that result. Mr. Richart's careful analysis and interpretation of our data brings out points and relations which were not brought out in such detail or clarity in the original paper. The mortar voids method of Messrs. Talbot and Richart is an extremely lucid and convenient method for proportioning mortars and concrete. I expressed my appreciation of the method when it was first presented to this Society. It is a method deserving of wider application and greater popularity. When one has to do with the "*post mortem*" examination it is necessary to find the volume of mortar, the volume of coarse aggregate and the quantity of cement per unit volume of concrete. The mortar voids method is the only method for proportioning concrete which sets up precisely these quantities in advance. If all concrete were proportioned by this method and accurate records were kept, there would be little necessity for "*post mortem*" work on concrete, and if such necessity did occur it would be a matter of considerable simplicity to compare the properties of the finished product with the properties of the concrete which should have resulted from proportions set up by the mortar-voids method. The application of the mortar-voids method in practice forces one to think in terms of quantities of the solid materials, water and air in the unit volume of concrete and it requires one to form a mental picture of the structure of the mass of concrete. These are very helpful and useful considerations, especially when taken in relation to other properties of the concrete such as strength and permeability.

THE FIRE RESISTANCE OF GYPSUM PARTITIONS[1]

By S. H. Ingberg[2]

Synopsis

Gypsum block partitions have been in use in this country for about 25 years. During this period fire tests followed by hose stream applications have been made of various gypsum block constructions at Columbia University for the New York Bureau of Buildings, at the Underwriters' Laboratories, Chicago, Ill., and by the British Fire Prevention Committee, the results of which are summarized in this paper. Gypsum blocks are either solid or hollow and are now generally made of plaster of Paris mixed with a small percentage of wood fiber and an excess of water which on drying leaves the block light and porous. Other materials that have been added to the plaster of Paris in varying amounts are cocoa and asbestos fiber, wood chips, coke breeze, cinders, ashes, lime, cork dust, pumice, and sand.

In the tests, gypsum block constructions of proper material and thickness gave performance indicating good ability to stop or retard fires in buildings, as used in non-bearing partitions not subject to heavy impacts. The temperatures transmitted to the unexposed side were in no case excessive and there was no pronounced bulging or cracking of the partition. The gypsum on the fire side dehydrated and became weak and crumbly. The water streams washed away this weakened material often exposing the air cells of the hollow blocks. Plaster that remained in place during the fire exposure was found to add considerably to the resistance of the partition and indicated the desirability of the present practice of scoring the surface of the blocks to give key for the plaster.

Introduction

Gypsum partitions built of solid or hollow block or tile units have been used for interior non-bearing partitions in American building practice during the past 25 years. There is evidence of use in Europe antedating this period by about 15 years. The main binding constituent in the blocks is re-hydrated calcium sulfate or gypsum ($CaSO_4 \cdot 2H_2O$) formed from the hemi-hydrate or plaster of Paris ($CaSO_4 \cdot \frac{1}{2}H_2O$), in the process of manufacturing the blocks. Aggregates that have been added include wood, cocoa, and asbestos fiber, wood chips, coke breeze, cinders, ashes, lime, cork dust, pumice, and sand. Minor amounts of liquid cements and glues have also been added. As reinforcement in the blocks, reeds and strips of wood have been used and in the joints between the blocks and through

[1] Published by permission of the Director of the Bureau of Standards, U. S. Department of Commerce.

[2] Physicist, U. S. Bureau of Standards.

adjacent blocks, dowels and bars of wood and metal, strips of wire mesh, or continuous metal wires or bars. To receive these dowels and bars and also to improve the mortar bond between the blocks their edges were grooved or channeled or the blocks were formed with holes for receiving the dowels. The blocks for use in partitions have been of rectangular shape from 10 to 16 in. high as set in the wall and from one to five feet long. The thickness for the solid block has ranged from $1\frac{3}{4}$ to 5 in. and for the hollow block from 2 to 6 in. The cores for the latter have been of various shapes, round, elliptical, and rectangular, with the holes generally placed horizontal in the wall, although a few

TABLE I.—STRENGTH, ABSORPTION AND WEIGHT OF GYPSUM BLOCK.

Testing Institution	Year of Test	Composition of Block	Thickness and Design	Compressive Strength lb. per sq. in.			Average Absorption, per cent	Average Weight lb. per cu. ft.
				Maximum	Minimum	Average		
Columbia University	1901	1:2 gypsum and ashes by volume	2-in. solid	...	...	325	..	78
	1901	" " " "	3-in. solid	...	...	490	..	70
	1901	" " " "	4-in hollow	...	...	243	..	66
	1906	1:3 gypsum and screened cinders by volume	$2\frac{1}{2}$-in. solid	560	117	...	35	..
	1906	" " " "	4-in. hollow	299	97	...	30	..
	1912	1:1 gypsum and steamed cinders by weight	2-in. solid	...	...	522	..	78
	1912	" " " "	3-in. solid	...	...	486	..	80
	1906	Gypsum 88, excelsior 9, shavings 3, by weight	2-in. solid	492	194	...	57	54
	1906	" " " "	3-in. hollow	153	96	...	57	39
Underwriters' Laboratories	1916	Gypsum and wood fiber (proportions not given)	2-in. solid	...	...	559	49	55
	1910–1918	" " " "	3-in. hollow	374	98	165	61	40
	1910–1918	" " " "	4-in. hollow	324	93	145	58	38
	1910–1918	" " " "	5-in. hollow	268	108	179	..	38
	1910–1918	" " " "	5-in. solid	1012	499	830	..	56
	1910–1918	" " " "	6-in.hollow	207	69	158	58	34
Bureau of Standards	1923–1924	Gypsum with 0.65 to 2.12 per cent wood fiber	2-in. solid	715	377	478	53	56
	1923–1924	" " " "	3-in. hollow	289	109	164	53	39
	1923–1924	" " " "	4-in. hollow	297	81	148	55	39
	1923–1924	" " " "	6-in. hollow	192	119	150	57	38

designs have been used for setting the blocks with the holes vertically in the wall. The treatment of the outside surface for reception of plaster has varied from leaving the surface smooth to deep mortised grooves.

During the past 15 years, considerable simplification in point of constituent materials, and design and size of blocks has developed. While blocks made with some of the early and now unusual combinations of materials are still in use, the materials used at present are quite uniformly unretarded first-settle gypsum (plaster of Paris) mixed with from 0.75 to 2.5 per cent of wood fiber (by weight). In the earlier gypsum wood-fibered blocks, the weight of wood fiber, excelsior or chips was up to 25 per cent of the weight of the gypsum. As now made, an excess of mixing water equal to from 60 to 90 per

cent of the dry weight of the gypsum is generally used. Since only from 15 to 19 per cent of water is needed for hydrating the gypsum, the excess water on being evaporated in the drying process that follows the molding, makes the block lighter and more porous than the normally set plaster. Other than the wood fiber no reinforcement is used in the blocks nor are reinforcements used in the joints in laying up the partitions except in cases where heavy partitions for shaft enclosures or fire division purposes are required to develop resistance against impact from falling building members or equipment. The cores at present in use are round, elliptical and rectangular, the rectangular cores having rounded corners. The treatment of the surface for receiving plaster usually consists of shallow horizontal grooves or closely spaced pits and projections, the object being to obtain the desired key for the plaster with a minimum sacrifice of net bearing area for the block.

COMPRESSIVE STRENGTH, ABSORPTION, AND WEIGHT

Table I gives a summary of tests made to determine strength, absorption and weight of typical gypsum blocks. Those made at Columbia University and the Underwriters' Laboratories were of block passing successfully the fire tests. For comparison there are given results of strength, absorption and weight determinations recently made at the Bureau of Standards and reported at the 1924 meeting of this Society.[1] The values given are the average or range of results, each figure representing from one to fifty tests. The compression tests were made with the block on edge either normally dry or oven dry, the unit loads being computed as for the gross bearing area, and the weight per cubic foot refers to the total volume of the block, inclusive of core spaces. The absorption is the percentage weight of water absorbed by the dry blocks when immersed for 2 hours or more at room temperature.

It will be noted that the solid gypsum and cinder blocks had strengths comparable with the solid gypsum and wood fiber blocks. The former weigh 70 to 80 and the latter 54 to 56 pounds per cubic foot. The hollow cinder blocks are somewhat stronger than the wood fibered blocks, probably due to a smaller core space. Since the weight of set gypsum having an absorption of 10 to 15 per cent is about 80 lb. per cu. ft., the decrease in weight to the 55 lb. for the wood fibered gypsum block materials is due mainly to the porosity resulting from the excess water used in mixing, as previously indicated. The

[1] J. M. Porter, "Properties of Gypsum Tile," *Proceedings*, Am. Soc. Testing Mats., Vol. 24, Part II, p. 901 (1924).

strength of wood fibered blocks tested saturated with water ranged from 20 to 70 per cent of that of similar blocks tested dry. It might be noted that the average strength, weight, and absorption of the blocks tested in the period 1910 to 1918 are approximately the same as for blocks of comparable design and thickness in the 1923 to 1924 series.

RESULTS OF FIRE TESTS

Fire tests of gypsum block partitions were conducted at Columbia University for the New York Bureau of Buildings, 18 reports of tests by Messrs. Ira H. Woolson and J. S. MacGregor being on record for the period 1901 to 1912. The Underwriters' Laboratories made 11 fire tests of gypsum block constructions during the years 1909 to 1918, and seven tests were made by the British Fire Prevention Committee from 1899 to 1905. This apparently covers all fire tests of gypsum block constructions made by these institutions and comprises all the data on the subject that is conveniently available. It covers fairly adequately the full range in block design and materials that have had any extended use in this country, and some of those tested apparently did not go beyond the development stage.

Columbia University Tests:

The furnace consisted of a brick hut into the two long sides of which were built the partitions to be tested, each test being usually made on two partitions of different thickness or design. The exposed length of the partitions was 14 ft. The exposed height was about 9 ft. The fire exposure was obtained from a wood fire on a raised grate, the top of which defined the lower line of exposure for the test panel. The firing was regulated so that an average temperature within the chamber of approximately 927° C. (1700° F.) obtained 30 minutes following a quick start, after which it was maintained as near this point as possible for 30 minutes, the duration of the test being one hour. At the end of the fire test a hose stream was applied to the exposed side of the partitions for two minutes, through door openings in the short sides of the test hut, the nozzle diameter of the playpipe being $1\frac{1}{8}$ in. and the pressure at the hydrant from 25 to 40 lb. per sq. in. This probably gave a nozzle pressure between 20 and 30 lb. In some of the later tests the pressure was gaged at the nozzle and maintained at 30 lb. The temperatures on the unexposed surface were measured at the middle of each panel with a thermometer whose bulb was in contact with the panel, the stem of the thermometer and the wall area around it being covered with an empty cigar box with the open side against the panel. In the tests conducted after 1910

the cigar box was replaced by a 2 by 4 by 4-in. asbestos pad. The thermometer was inserted through an inclined hole in the pad and had its bulb in contact with the unexposed side of the panel.

Plaster-Cinder Block Partitions.—Eight tests were made with plaster and cinder block partitions, the proportions of the mixture for the blocks being from 1:1 to 1:3 volume parts of plaster of Paris and cinders. In one case lime was added to the mixture and in another a small amount of liquid cement. They were laid up in gypsum or portland cement - lime mortar. Plaster, $\frac{1}{4}$ to $\frac{3}{4}$ in. in thickness was applied to each side after erection, the most usual thickness being $\frac{1}{2}$ in. The partitions were tested at an average age of 9 days after the plaster was applied, the range being from 5 to 20 days.

In five of the tests, the plaster fell off from one half to the full exposed area during the first thirty minutes of the fire test. The blocks in four of these partitions were not scored; for the other the records gave no information relative to scoring. On one partition that had plain shallow groove scoring and on another that had deep indentations the plaster remained in place until the end of the test. Some fine cracks in the blocks and joints appeared on the unexposed side but they were not of serious account. Maximum deflections of $\frac{1}{2}$ in. were noted for two partitions. The indicated surface temperatures did not in any case exceed 100° C. (212° F.) the range being from 60 to 99° C. (140 to 210° F.).

For the cases where the plaster fell off during the fire test, the hose stream application at its conclusion eroded the area on which it was applied to a depth of $\frac{1}{2}$ to $1\frac{3}{4}$ in. The cells were generally exposed for the hollow blocks. For the partitions that did not lose their plaster in the fire test, the water washed off part of the plaster and only slightly pitted some of the blocks. None of the eight partitions collapsed or had holes through them from the water application.

Plaster-Ash Block Partitions.—Three tests were made of $2\frac{1}{2}$-in. hollow and 2 and 3-in. solid block partitions, the mixture proportions for the blocks, where given, being 1:2 parts by volume of plaster of Paris and ashes, with minor additions of lime or liquid cement. They were all plastered on both sides. There are no notes as to the degree to which the plaster remained in place during the fire test, except in one case where only the thin finish coat came off. The water application washed off most of the plaster and eroded the blocks to depths of $\frac{1}{4}$ to $\frac{1}{2}$ in. Some of the cells were exposed on the hollow blocks. All the partitions passed the test requirements.

Plaster-Fiber Block Partitions.—Six tests were made with partitions where the blocks were of plaster of Paris and wood fiber or

excelsior in proportions by weight ranging from 10:1 to 3:1. In one case a minor amount of asbestos was present and in another the asbestos fiber was a little in excess of the wood fiber. In another test the block composition was 1:1:1 plaster of Paris, wood fiber, and infusorial earth, with some carbonate of lime added. This was plastered with lime plaster gaged with gypsum. All the other partitions were plastered on both sides to an average thickness of $\frac{1}{2}$ in. with what is stated to be gypsum plaster, where records give information. The thickness of the solid blocks was $1\frac{3}{4}$ and 2 in. and the hollow blocks 3 and 4 in., giving total partition thicknesses of $2\frac{3}{4}$ to 5 in.

The test characteristics were about the same as with previous groups although the records do not give as much information on the extent to which the plaster remained in place during the fire test. One partition of $1\frac{3}{4}$-in. solid blocks deflected 4 in. during the fire test and collapsed on application of water. This had a wood and cocoa fiber content of 22 per cent. A partition of the same material but with 2-in. thick blocks passed the same test successfully. Deflections for some of the other partitions are reported to have been from $\frac{3}{4}$ to $1\frac{1}{4}$ in. The water application washed away the exposed area to a depth of $\frac{1}{4}$ to 1 in. and exposed the cells of the hollow units over $\frac{2}{3}$ or more of the area for three of the partitions.

Tests at the Underwriters' Laboratories:

The furnaces used at the Underwriters' Laboratories were gas-fired, with a shallow vertical chamber, the test panel being contained within a frame that was rolled in front of the furnace for the test and thus formed one wall of the chamber. Temperatures on the unexposed surface were measured at a number of points with thermometers whose bulbs were attached to the surface with plaster of Paris which covered them and the stems for about $1\frac{1}{2}$ in. to a depth of about $\frac{1}{4}$ in. The fire test was continued for two hours when the wall was pulled away from the furnace and a hose stream applied for five minutes to the sides exposed to the fire, using a playpipe with $\frac{7}{8}$-in. nozzle, under water pressure of 50 lb. per sq. in., located about 20 ft. from the panel.

The blocks in these tests were all made of plaster of Paris and wood fiber. The amount of wood fiber is not given but it was probably below the average for the wood fibered blocks in the Columbia University series.

Tests of Plastered 3-in. Hollow Block Partitions.—Nine fire tests were made with 3-in. hollow blocks plastered on both sides with gypsum plaster giving a finished partition thickness of 4 to $4\frac{1}{2}$ in. The panels were about 7 ft. wide and 9 ft. high and were built solidly into the containing frame. The fire exposure was regulated so that

an indicated furnace temperature of about 1700° F. was obtained at 30 minutes, after which the fire was maintained to give approximately this temperature up to the end of the 2-hour fire test. The plastered panels had aged from 23 days to 5 weeks when subjected to fire test.

During the fire test fine cracks formed on the unexposed side through the blocks and in the joints, but did not open up sufficiently to pass flame or any considerable amount of smoke. The plaster on the fire side in some tests fell during the fire test and in others remained in place. While the results do not give a basis for definite conclusions, there are indications that adequate scoring helped to key the plaster in place. The exposed side was checked and dehydrated by the fire to depths of 1 to $1\frac{3}{4}$ in. No deflections reported were greater than $\frac{1}{2}$ in. The indicated surface temperatures at the end of the test ranged from 52 to 80° C. (125 to 175° F.).

The water application washed off all the plaster and the calcined exposed part of the panel, exposing the cell spaces on from one-half to the total area of the panel. In one test the hose stream made a hole through the partition at the end of one minute. In this test the plaster had all fallen off during the fire test. The average compressive strength of the blocks not exposed to fire was 70 lb. per sq. in. In a later test with blocks having an average compressive strength of 106 lb. per sq. in. the panel withstood both fire and water tests successfully. Only the finish coat of plaster spalled during the fire test. The water eroded the blocks to a depth of $1\frac{1}{4}$ to 2 in.

Another partition failed in the water test by collapse of one-half of the partition. Only the finish coat had spalled during the fire test and the maximum deflection was $\frac{3}{4}$ in. The compressive strength of the blocks was ample, being 200 lb. per sq. in. on the average, but the air spaces were formed by elliptical cores which left thin exterior and interior webs. On application of water, the exposed side and these thin webs were washed away leaving the remainder of the partition too unstable to withstand the impact of the hose stream. In a later test with heavier webs, the partition successfully passed the test requirements.

Test of 3-in. Hollow Unplastered Partition.—The partition was built up of 3-in. hollow wood-fibered blocks laid up in 1:3 gypsum mortar. No plaster was applied. The partition was 10 ft. wide and 11 ft. high and was tested two days after it was laid up. It was subjected to a fire test on one side for 2 hours and 15 minutes, the furnace exposure being regulated to conform with the standard time-temperature curve defined in the Standard Specifications for Fire Tests

of Materials and Construction (C 19 – 18) of this Society. During the fire test, fine horizontal cracks began to appear on the outside near the end of the first hour. At one hour and 40 minutes the furnace glow could be seen through openings in joints between the blocks which later emitted hot gases and flames. At 2 hours a marked vertical crack developed near the middle of the unexposed side which opened up to a width of $\frac{3}{8}$ in. near the end of the test. The maximum deflection was $\frac{3}{4}$ in. away from the fire. The average indicated temperature on the unexposed side was 71° C. (160° F.) at 1 hour 55 minutes. It had not risen more than 17° C. for $1\frac{1}{2}$ hours. At 2 hours 20 minutes, the partition collapsed because of weakness induced by calcination of the blocks. The fire and water test was made on a separate duplicate partition which was subjected to the standard fire test for $\frac{3}{4}$ hour, after which it was immediately withdrawn from the furnace and a hose stream applied to the heated face through a $\frac{7}{8}$-in. nozzle under a water pressure of 50 lb. per sq. in. During the fire test the exposed side of the blocks was calcined to a depth of $\frac{1}{2}$ to $\frac{7}{8}$ in. The maximum deflection was $\frac{1}{2}$ in. toward the fire. The water carried away most of the calcined material and exposed the cells over a few square feet of the partition. After cooling, the ends of the partition were cut free from the containing frame and the partition subjected to a transverse center load, failure occurring at a total load of 26 lb., which is equivalent to a uniformly distributed load of about $\frac{1}{2}$ lb. per sq. ft.

Test of 5-in. Solid Reinforced Partition.—The blocks were 5 in. thick, 24 in. long and 12 in. high with ends grooved to receive $\frac{1}{2}$-in. vertical reinforcing bars which were grouted in with the 1:3 gypsum and sand mortar in which the blocks were laid up. A 1:2 gypsum and sand plaster was applied to both sides, which, inclusive of the white finish, had an average thickness of $\frac{3}{4}$ in., giving a total partition thickness of $6\frac{1}{2}$ in. The blocks were about 75 days and the plaster coatings 29 days old when the partition was subjected to the fire test. The height of the partition was 11 ft. and the width 10 ft. An average furnace temperature of 1060° C. (1940° F.) obtained at the end of the first hour and 1260° C. (2300° F.) near the end of the test. This is a more severe exposure than that for the other tests previously described. The firing was continued for $4\frac{1}{2}$ hours, when the panel was withdrawn and allowed to cool. The full fire resistance of the partition had not been developed. Only a few fine cracks had formed on the unexposed side during the fire test. A maximum deflection toward the fire of $\frac{1}{2}$ in. occurred at 30 minutes. Except for flaking of the finish coat no plaster fell off in the fire test. The average indicated

temperature of the unexposed side was near 66° C. (150° F.) during the last 3 hours of the fire test. The blocks were checked and calcined to about half depth. After cooling, the panel withstood a lateral load equivalent to about 10 lb. per sq. ft. before failure. A duplicate partition was subjected to the standard fire exposure for ¾ hour when it was withdrawn and a hose stream from a 1⅛-in. nozzle under 50 lb. per sq. in. water pressure was applied over the exposed face for 2½ minutes. The plaster and block material calcined during the fire exposure was washed away to a depth of about 1¼ in. The ends of the partition were later cut loose from the panel frame and loaded laterally with a center load of 2525 lb. when failure occurred. This is equivalent to a uniform load of 50 lb. per sq. ft. and indicates a good degree of stability after fire and hose stream applications.

Calcination Tests.—A series of tests to determine the effect of heat of different intensities and durations on gypsum block material was conducted by the Underwriters' Laboratories in 1911. Solid blocks 2 ft. square and 6 in. thick made of calcined gypsum (plaster of Paris) having a calcium sulfate content of about 91 per cent, hydrated with water and having an addition of about 4.5 per cent of wood fiber, were used for the tests. The block to be tested was mounted in a movable frame and placed to form one wall of a small gas-fired furnace. The test durations were ½, 1, 1½, 2, 3, and 4 hours, and the temperatures maintained constant during the test period were 1000, 1300, 1600, 1900, and 2200° F. One test at each temperature for each duration period was made, making a total of 30 tests. The temperatures were measured at 1-in. intervals throughout the thickness of the block for the tests with durations of 3 and 4 hours. The depth of calcination was determined visually and by means of a needle penetrometer which was weighted so that it would indicate calcination or injury to the material with penetrations exceeding the 0.03 to 0.04 in. obtained with the unexposed gypsum block. Other items determined were depth to which the wood fiber was consumed or carbonized, the depth of surface hardening or vitrification, the depth and width of shrinkage cracks, the shrinkage in thickness and the estimated effective depth of injury, which was taken as equal to the depth of calcination plus the shrinkage.

With furnace temperature of 1000° F. and duration of exposure of ½ hour and 4 hours, the depth of calcination was ⅝ in. and $1\frac{15}{16}$ in. for the two extreme exposure limits, respectively. The wood fiber was consumed and the blocks were checked for depths from ⅜ in. at the one limit to 1¼ in. at the other. The checks were from $\frac{1}{64}$ to $\frac{1}{32}$ in. wide. There was very little surface hardening or

shrinkage in the thickness of the blocks. For exposures at 1300° F. these effects penetrated about 25 per cent further into the block. Where exposed at 1600° F., these effects were further increased and surface hardening or vitrification and shrinkage up to $\frac{1}{8}$ in. in the thickness of the block was observed. For exposures at 1900° F. the calcination extended to depths of 1 and 3 in. for respective durations of $\frac{1}{2}$ hour and 4 hours; the fiber was carbonized and blocks checked for depths from $\frac{3}{4}$ to $2\frac{3}{8}$ in., the width of the checks ranging from $\frac{1}{64}$ to $\frac{7}{32}$ in. The shrinkage in thickness was $\frac{3}{32}$ in. for the $\frac{1}{2}$-hour exposure and $\frac{9}{16}$ in. for the 4-hour exposure, giving effective depths of injury of $1\frac{3}{32}$ and $3\frac{9}{16}$ in. The depth of vitrification ranged from $\frac{1}{8}$ to $\frac{7}{8}$ in. For the 2200° F. exposure for 4 hours the calcination extended to a depth of $3\frac{5}{16}$ in., which, with a shrinkage of $\frac{7}{8}$ in., gave an estimated depth of injury of $4\frac{3}{16}$ in. The material on the exposed side was vitrified to a maximum depth of one inch.

The temperatures in the blocks one inch from the fire side rose to 100° C. (212° F.) within 10 minutes and increased thereafter at a rate dependent on the exposure temperature, being at 4 hours from one-half to two-thirds of the furnace temperature, except in the test at 2200° F. where it was 1082° C. (1980° F.). This rapid rise was caused by the checking of the material. At 2 in. from the fire side, a temperature of 100° C. (212° F.) was exceeded at 2 hours, only for exposure temperature 2200° F. when it was 158° C. (316° F.). At 4 hours the temperature at this point was 107° C. (224° F.) for exposure of 1000° F., and 679° C. (1255° F.) with exposure temperature of 2200° F. At points 3 in. from the fire side the temperature of 100° C. (212° F.) obtained only with exposure temperatures of 1900 and 2200° F. and then only during the last hour, being at 4 hours 108° C. (226° F.) for the 1900 and 158° C. (315° F.) for the 2200° F. exposure. At 4 in. and 5 in. from the fire side, 100° C. (212° F.) was exceeded only by a few degrees at 4 hours for the highest exposure temperature. The temperature on the unexposed surface at 4 hours was 70° C. (160° F.) for the 1000° F. exposure and 98° C. (208° F.) for the highest (2200° F.) exposure temperature.

Other calcination tests were conducted in connection with the earlier partition tests. One series of tests at temperatures from 175 to 250° F. (80 to 121° C.) indicate that calcination effects sufficient to weaken the block occur at these temperatures if the exposure is sufficiently prolonged. The temperature limit below which such effects do not take place is difficult to define, since it depends on the humidity, and the constancy of both temperature and humidity. If the variations are sufficient to allow for re-hydration at intervals, the

temperatures can go higher than for constant conditions. However, since in rooms, such as drying rooms, maintained at temperatures above normal, the relative humidity will always be below normal unless special measures are provided, it appears that gypsum partitions should not be used where the usual temperatures over long periods appreciably exceed those of the surroundings.

Tests by the British Fire Prevention Committee:

The partition to be tested was built between the walls of a test hut 10 by 10 ft. in inside lateral dimensions, the partition height being $7\frac{1}{2}$ to 9 ft. The furnace chamber thus formed on one side was about 7 ft. deep and the passage way between the wall of the hut and the unexposed side of the partition, about 2 ft. In the middle of the end wall of the hut was placed an open doorway, 2 ft. wide by $6\frac{1}{2}$ ft. high, which gave access to the unexposed side of the partition. The fuel used for the fire exposure was producer gas supplied to the furnace chamber through mixing chambers with checkerwork of fire brick. The firing was regulated to obtain a minimum temperature at the end of the test of 815° C. (1500° F.) for test durations of one hour or less, and 982° C. (1800° F.) for tests of longer duration. These minimums were generally exceeded by 100 to 300° C. The temperature rise during the first stages of the test was less rapid than defined in the Standard Specifications for Fire Tests of Materials and Construction (C 19 – 18) of this Society, being retarded by the brick checkerwork through which the burning gases entered the furnace chambers. Temperatures on the unexposed side were measured with thermometers hung in the 2-ft. passage and also, in some tests, with thermometers in contact with the partition. The age of the partitions at the time of the test was from 10 to 30 days, a slow coke fire being maintained in the furnace chamber for 3 to 14 days before the test.

The composition of the gypsum blocks presented a variety of combinations. In two tests that might be classed together, calcined gypsum was mixed with hydrated lime and asbestos fiber with some sulfuric acid added to the mixing water for the hollow blocks of the one partition, and for the other solid blocks were made of approximately equal parts of calcined gypsum, pumice, hydrated lime, and sand, and had $\frac{5}{16}$-in. vertical iron bar reinforcements on 16-in. centers in grooves on the face of the partition. The partitions were plastered with thin coatings of fire clay in the one case and portland cement and sand plaster in the other, the total thickness of each partition being $3\frac{1}{2}$ in. The first partition was tested to one hour with maximum temperature of 1100° C. (2000° F.) after which a hose stream from a $\frac{1}{2}$-in. nozzle at 20 lb. pressure was applied over the partition for 2

minutes. No cracks or other developments of note occurred during the fire test. At 30 minutes the outside face of the partition was too hot for the hand. The hose stream only washed off the fire clay coating. The other partition was subjected to a $1\frac{1}{4}$ hour fire test with hose stream application for 2 minutes from a $\frac{1}{2}$-in. nozzle at 40 lb. pressure. The furnace temperature at the end of the test was 1060° C. (1940° F.). The water application washed away most of the plaster and exposed the reinforcement to some extent.

In another test the blocks were made of calcined gypsum, cocoa fiber and cork dust and reinforced with reeds. The blocks were solid and $2\frac{3}{4}$ in. thick, the fire side being heavily scored and plastered with a 1-in. thickness of 1:3 gypsum and sand plaster applied in two coats. The other side was lightly scored and given a similar coating $\frac{1}{4}$ in. thick. During the fire test of $1\frac{1}{4}$ hours' duration the outer plaster coat on the fire side fell off and the reeds nearest to the fire charred. Only fine cracks were visible on the unexposed side. The water application through a $\frac{1}{2}$-in. nozzle at 20 lb. for 2 minutes washed away most of the plaster but otherwise had no effect.

In a fire test of $1\frac{1}{4}$ hours' duration with a $2\frac{3}{4}$-in. solid unplastered partition, the hose stream applied at its conclusion through a $\frac{3}{4}$-in. nozzle at 55 lb. pressure for 2 minutes eroded the blocks for $\frac{7}{8}$ in. depth and the mortar joints for a greater depth. The partition did not collapse and was rated as passing the test satisfactorily. The blocks had tongue and groove edges, which provided a stop that prevented the mortar between the blocks from being completely washed out.

A partition of $2\frac{1}{4}$-in. solid blocks made of equal parts of calcined gypsum and coke breeze (screenings) reinforced in vertical and horizontal joints with $\frac{1}{16}$-in. wires and plastered with $\frac{1}{4}$-in. coatings consisting of equal parts of gypsum and coke dust, was subjected to a $1\frac{1}{2}$-hour fire test. A considerable portion of the plaster fell off during the fire test, and at its conclusion, a hose stream through a $\frac{3}{4}$-in. nozzle at 40 lb. applied for 2 minutes greatly eroded the blocks and made a 3 by 6-in. hole through the partition at one point.

For another test, solid plaster blocks, 2 in. thick and reinforced during erection with $\frac{1}{4}$ by $\frac{1}{2}$-in. wood strips set vertically in the joints and in holes in the block, and grouted in place with gypsum plaster, were given a $\frac{1}{8}$-in. coat of plaster on the fire side. The fire test was continued for $2\frac{1}{2}$ hours, the maximum temperature indicated by thermometers against the outside face of the partition being 104° C. (220° F.). Most of the thin plaster coat fell off and the partition deflected away from the fire $\frac{3}{4}$ in. at 2 hours. On application of the

hose stream through a $\frac{3}{4}$-in. nozzle for 2 minutes, 20 sq. ft. of the partition fell in. The rest was eroded to a depth of $\frac{1}{2}$ in. The wood strip reinforcement had been completely carbonized. In a later test, the thickness of the blocks was increased to $2\frac{3}{8}$ in. and $\frac{1}{4}$-in iron tubes were substituted for the wood strips. During the fire test, cracks formed extending to the unexposed side. At 1 hour 50 minutes, the glow of the fire could be seen through the cracks and some smoke was passed. The temperature in the passage next to the unexposed side was 138° C. (280° F.). The partition deflected $4\frac{3}{8}$ in. toward the fire. The water application from a $\frac{3}{4}$-in nozzle for 2 minutes washed away the plaster and the block material, particularly where cracks had formed during the fire test, openings up to $\frac{1}{4}$ in. wide being formed at the joints. The water pressure used in this and the preceding test is not stated but it probably was near 40 lb., since this was the pressure used in partition tests preceding and following those here described.

DISCUSSION AND SUMMARY

The tests, the results of which have been outlined, cover a wide range in materials, design and methods of test, and while definite information was not obtained on some points, there are a number of general facts that the tests can be considered to have established.

Constituent Materials:

The gypsum-cinder and ash combinations that marked the initial use of gypsum block partitions in this country had, with proper thickness and design, sufficient strength, fire resistance and ability to withstand the erosion of hose streams to serve the purposes for which non-bearing interior room and corridor partitions are used. The same would apply where pumice is used for aggregate, and also for minor additions of lime, kieselguhr, and sand. They had greater weight as compared with the gypsum and wood fiber blocks, which without doubt restricted their use. It should be noted that this paper is not concerned with any property of the construction that is not directly related to fire resistance. These other properties which might include ease of installation and plastering, freedom from tendency to stain or otherwise deteriorate the finish and ability to resist attack of vermin such as mice and rats, are often determining features, but are not directly related to fire resistance.

The earlier combinations of gypsum with large percentages of wood fiber gave blocks of fair strength and fire resistance and were lighter than the gypsum cinder blocks. The large percentage of fiber without doubt increased the transverse strength of the blocks and

made them better able to withstand shipment and handling. When exposed to fire, the combustible fibers on the fire side rapidly lose strength and carbonize, although back of the zone of calcination their presence would strengthen the partition. The substitution for wood fiber of asbestos fiber of comparable length undoubedly increases the stability of the partition under fire conditions, although the complete dehydration and checking of the gypsum, which was shown by the calcination tests to progress to about the same depth as the carbonization of the wood fiber, limit the ability of even incombustible fibers or reinforcements to add strength to the side exposed to the fire. Large amounts of combustible materials such as were present in the 1:1 gypsum and coke breeze partition, and combustible reinforcements in the form of wood strips or reeds did not give favorable results. When present in such large amounts and in continuous strips, they carbonize or burn more readily than when mixed with a preponderance of incombustible materials.

In point of relative merit of the larger percentages of wood fiber used in the earlier constructions as compared with present practice, the tests indicate that the small percentages at present in use give at least equal resistance. The partitions tested by Columbia University in which the larger amounts of wood fiber were present, had shorter fire exposures and hose stream applications than those tested at the Underwriters' Laboratories, which had a smaller wood fiber content. The fire effects such as cracking and deflection were about the same for the two series. In none of the tests did the temperatures recorded for the unexposed side exceed 100° C. (212° F.). The effects of the water were a little more severe for the tests at the Underwriters' Laboratories which can be ascribed to the longer fire exposure preceding it and the greater volume and pressure of water applied.

Compressive Strength:

The tests give indications that a certain amount of hardness is necessary for stability after portions of the blocks have been calcined by fire exposure and hose streams are applied. Where blocks are used without plaster it would appear desirable that they should be fully adequate in this particular. Hard aggregates such as cinders, and sand would apparently increase resistance in this particular and also decrease shrinkage and checking.

Design of Block:

The desirability of scoring to help the plaster remain in place during the fire exposure has already been indicated. Gypsum plaster should preferably be used on gypsum blocks as it gives the best adhesion under fire conditions. The tests have also shown that

the shells and webs cannot be reduced beyond a certain minimum if the necessary stability during fire and water exposure is to be maintained. The tongue and groove design for the edges appears desirable where the blocks are used without plaster. In the tests, one unplastered partition would apparently not have passed the test requirements without this detail and another would have passed with a better margin if it had been used.

Effectiveness of Reinforcement and Ties:

For use in ordinary room partitions there appears no need for the rather elaborate systems of tying and dowelling used in some of the earlier constructions. The blocks should, however, be set in the wall so as to avoid having successive vertical joints on the same line. Gypsum blocks should be set in gypsum and sand mortar as rich as it can be worked, which will be in volume proportions of 1:2 or 1:3. Where gypsum blocks are used for fire division purposes for enclosing stairways and elevators and in all locations where impacts are to be expected, greater thicknesses than those used for ordinary partitions are necessary for stability, and continuous reinforcements tied into the supports are desirable. Where the fire exposures and impacts that can be developed are very severe, the solid block construction with suitable reinforcements should be used.

Temperatures on the Unexposed Side:

As previously indicated, the temperatures on the side away from the fire in all of the tests where measurements were made, were not excessively high. While these temperatures refer to the free surface and would have been considerably higher if measured under pads representative of materials stored against the unexposed side, it appears that the fire resistance of ordinary gypsum partitions is not limited by the temperatures developed on the unexposed side but rather by the length of time required to calcine the blocks to such depth as to impair seriously the stability of the construction. The factors on which the general fire resistive properties of gypsum depend have been discussed in a previous paper.[1]

Deflection and Cracking:

The deflections of gypsum partitions due to temperature differences between the two sides are quite moderate. Deflections toward the fire are to be expected during the first part of the fire test, but later the shrinkage on the fire side more than balances the expansion and the final deflection is often away from the fire. The maximum

[1] S. H. Ingberg, "The Fire Resistive Properties of Gypsum," *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part I, p. 254 (1923).

deflections during one or two-hour fire tests seldom exceed one inch for total partition thicknesses of 3 in. or over. Fine cracks usually form on the cold side during the first part of the fire test, through the blocks and in the joints between them. Some of these cracks do not extend deeper than through the outside plaster. For plastered constructions these seldom open up sufficiently to pass hot gases, fire or smoke in such amount as to limit the fire resistance. In the tests where such deflections, cracks and openings became of objectionable size, the partition was too thin, the blocks were not made of proper materials, or other apparent defects in design were present.

Resistance to Water Application:

The water washes off the material calcined or damaged during the fire exposure and may erode the blocks considerably beyond this depth if the material is not sufficiently hard. Exterior and interior webs should be of adequate thickness in order that they may not be completely washed away along with the exposed shell. Where the plaster remained in place during a fire of one hour duration, hollow block partitions in many cases came through the hose stream application without loss of the shells on the fire side.

Limitations Imposed by Building Details:

Where gypsum partitions are built on combustible floors and have openings with wood framing, doors and sash, the fire resistance is limited by these details. Even so used, they serve the ordinary purposes of a partition and while no considerable degree of fire resistance is to be expected, the blocks add no fuel to the fire, and, as taken over the whole exposed area, they exert without doubt appreciable retarding effects.

To develop the full fire resistance of the construction, gypsum partitions should be erected on incombustible foundations, preferably directly on incombustible and fire resistive floor constructions and against similar constructions at the top and sides. Door and window openings, where present, should be suitably framed with incombustible materials and doors and sash should be of metal and glazed with wire glass. Framing for heavy doors should preferably be anchored to the floor or roof construction above, to relieve the partition from undue impacts. With such details, gypsum block partitions can be practically applied in building construction so that they will develop a large part of the fire resistance indicated by performance in tests.

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BRICK

By D. Knickerbacker Boyd[1]

Synopsis

The purpose of this paper is to show that any requirements for an individual brick should be based upon the performance expected of the collective units when assembled under actual service conditions in the form of a wall or pier rather than upon the performance of the unit itself. This inevitably brings us to a consideration of existing stipulated requirements or specifications for bricks and to the available records of tests on brick walls, piers and other structural features.

Requirements for individual brick are set forth in nearly every municipal or state building code. But they usually specify a hard, sound, well-burned brick. The National Board of Fire Underwriters and the Department of Commerce Building Code Committee have formulated requirements as suggested national standards for brick. The Department of Commerce Building Code Committee in two documents refers to the Specifications for Building Brick (C 21 – 20) of this Society, with modifications in each case which show the need for a thorough revision of these specifications, on which Committee C-3 itself has long been working.

Tests on brick piers, for strength, have been made at the Watertown Arsenal and in several educational institutions. Tests on brick walls, for fire resistance and other tests for strength have been made by the U. S. Bureau of Standards, and perhaps a few others (but all too few) have been made elsewhere. Some tests have been made for heat transmission through brick walls but none have ever been made on walls for absorption and for some other very important determinations.

In this paper an attempt is made to point out some of the knowledge concerning brickwork, as well as brick, which should be available through greater familiarity with actual structural requirements in order to formulate improvements upon any specifications for brick. Such specifications should be workable and elastic enough to provide for the selection of the different grades of brick common to each locality and suitable to appropriate conditions of use. With particular reference to the existing specifications of this Society, these would involve consideration of tentative revisions through some of the following ways:

1. To minimize the requirements for compressive strength.
2. To emphasize the requirements for transverse strength.
3. To greatly modify the requirements for absorption or to revise by fixing a strength value for the brick when saturated.
4. To classify the brick according to the two highest tests rather than the one lowest, with perhaps limits on that property which is to be least considered,

[1] Architect and Structural Standardist, Philadelphia, Pa.

rather than waiving same as is now provided for by at least one national authority.

5. To recommend methods of adjusting the specifications to suit variations in local clays and structural requirements.

6. To define the standard size as an approximation to be averaged.

In considering permanent changes in, or a complete revision of, the present specifications, an endeavor will be made to show that the following information should be obtained and fully considered:

First.—On how absorption affects brick in the wall, and on how much water a wall of given thickness should absorb in a given time.

Second.—On the strength and weight of walls wet and dry.

Third.—On the effectiveness of field tests for transverse strength as determining the general characteristics of a brick—A device for this purpose has been designed by the U. S. Bureau of Standards.

Fourth.—On the advisability of testing for surface hardness in addition to compression.

Fifth.—On the difference between bricks tested on edge and flatwise.

Sixth.—On the effectiveness of the sodium sulfate test as an indication of weathering qualities.

Seventh.—On the effect of alternate freezing and thawing of brick.

Eight.—On the reclassification of brick according to probable use rather than by arbitrary limits.

Ninth.—On the physical qualities of brick in actual masonry construction with different mortars and different bonds.

Due to the infinite variety of chemical compositions in clay itself and due to climatic and geologic conditions to which all clays, wherever found, have been subjected through the ages, it is impossible to obtain a raw material uniform in all localities for the manufacture of brick. Because of this intrinsic difference in the raw material, it is practically impossible to manufacture brick from different clays which will have the same physical properties. It would, therefore, seem impossible to arrive at one set of specifications which accurately and fairly cover the many variations in physical properties of the different kinds of brick produced throughout the country.

Some brick, for instance, which have throughout an entire region, been in satisfactory use for years as hard brick, have physical qualities which would, on the basis of the present A.S.T.M. Specifications, classify them as soft according to absorption, medium according to compression and hard according to modulus of rupture. It would assuredly be an economic waste to classify these bricks as soft when they have proved thoroughly serviceable in a climate subject to heavy rain and severe cold and to suggest the exclusion of such brick by architects, builders, manufacturers, and the public. Yet this is an actual situation which is presented when authorities in one part

of the country specify brick according to the A.S.T.M. Standards, for buildings in another part of the country.

Until such requirements are changed, I should like to see an explanation of the classification printed somewhat like the following:

"This classification is arranged from results of tests made on various bricks and is intended to be a hypothetical classification. Due to the variations in results obtained from tests on bricks manufactured from clays of different chemical composition, the requirements may be altered provided the resultant combination of physical properties has proved satisfactory in actual use in the locality intended and that the change is stipulated in specifications as a regional modification of the A.S.T.M. Standard."[1]

Practically, logically and scientifically there should be a separate set of specifications for bricks made from various kinds of clay and for their several uses. It is different with a strictly manufactured product for which it is necessary to standardize the various ingredients as to quality and particularly as to quantity and where it is impossible to characterize the product by visual inspection and physical demonstration "on the job." This is not the case with a brick, which by its color and ring, may, with a knowledge of local conditions, be generally determined as to whether it is a strong, permanent and thoroughly satisfactory unit for its intended use.

It is my hope that some day, in place of hard and fast dividing lines between compression, modulus or rupture, tensile strength and absorption, a chart and description of characteristics may be devised whereby the specifier may select the properties which he desires the brick to possess for the intended use and describe them according to grades locally obtainable. A specification would then read somewhat like this: "All brick shall be tested as prescribed in the A.S.T.M. Specifications for Building Brick, and shall fall within the following classifications: Exterior walls below ground floor level, A 75, B 50, C 65, D 15, commonly recognized as hard brick; above the ground floor level, for exterior face, the same; for backing up (so and so) known as medium brick; for certain interior walls (so and so) known as soft brick."

This is not an entirely new thought. One committee of the Society has already developed a scoring system for grading fabrics. But more relevant to the subject under discussion and somewhat analagous is the "Scheme of Service Classification of Refractories" developed by Committee C-8 on Refractories. This was published in the report of that committee for 1923[2] and will be the basis of

[1] The same explanation would be applicable to specifications for sand, gravel, broken stone and other products subject to local use.

[2] *Proceedings*, Am. Soc. Testing Mats., Vol. 23, Part I, p. 216 (1923).

further studies by the Sub-Committee on Clay Building Brick, of Committee C-3, of which the author is chairman.

With these remarks as a general introduction, I will discuss now some of the important considerations about tests for brickwork from the standpoint of actual use as compared with present Society requirements for individual units, under the following headings:

Compressive Strength	Absorption (including freezing and thawing)
Modulus of Rupture	Mortar and Bond
Tensile Strength	Size and Tolerances

Compressive Strength:

The compressive strength of a brick has usually been considered the chief index of the quality of the brick. And yet, in tests that have been made on walls, it has been found that failure has often been caused by breaking of the header bricks before crushing of the individual brick. The results of practically every test of which I have record tend to prove that the strength of the wall is limited by the modulus of rupture and perhaps by the tensile strength of the header bricks rather than by the compressive strength of either the header or stretcher units. Therefore, the vital factor in determining the strength of any piece of brickwork would seem to be the modulus of rupture.

The hardness of any material is usually emphasized as an indication of what may be expected in wearing qualities. However, the exposed surface of a brick in a wall is the part which is subjected to wear by the action of the elements and may therefore be the important part to consider. If it is definitely proved that the modulus of rupture limits the serviceable strength of the brick, it might be advisable to test a brick for surface hardness rather than for compressive strength. It very likely would be found that, for example, some brick which now are classified as medium would be perfectly satisfactory if classified as hard.

As to methods of testing for compression, some specifications call for testing brick on edge and some flatwise. It would certainly seem to be advisable to test all building units used in the same manner by similar methods. Information gained would then always be comparable.

Modulus of Rupture:

The modulus of rupture or transverse strength is a most important quality of the brick and strange as it may seem, many building codes completely ignore it. As has already been stated, the modulus of

rupture usually determines the failing point of a wall or pier, and by merely specifying compressive strength, as is often done, it might be that entirely the wrong deductions would be drawn as to the real serviceable strength of the wall. For instance, brick from the New York district might be compared with brick from the Illinois district as follows:[1]

	Modulus of Rupture, lb. per sq. in.	Compressive Strength, lb. per sq. in.
New York	601	5630
Illinois	1180	3200

The accepted method of estimating strength by the test for compression, which is used by architects, contractors, bricklayers and others would show the New York brick the stronger, whereas in reality the Illinois brick might develop greater strength in the wall. In support of this possibility the Federal Specification Board is considering specifications for brick which would require only modulus of rupture tests, which may be made in the field.

Absorption:

What does absorption mean? According to present practice, it means the amount of moisture which the brick can be forced to absorb. This information is all very well but just what "absorption" means so far as the actual use of the brick is concerned is another matter since the absorption may affect the building or wall in which the brick is used in at least five ways. It may

1. Weaken the brick and consequently the strength of the wall.

2. Add weight to each brick. It is advisable to know whether the absorption of the individual brick is sufficient to increase the dead weight of the wall to a dangerous degree.

3. Affect the amount of moisture which might penetrate to the inside of the wall. Such penetration would probably vary according to mortar, bond and thickness of the wall with the same brick. In practice, it has been proved that bricks with little or no absorption do not make as satisfactory a wall as bricks with a considerable percentage of absorption.

4. Affect the adhesion of the mortar. This would without doubt have a marked effect on the strength of the wall but has never been determined satisfactorily.

5. Affect the resistance of the wall to freezing and thawing.

[1] Figures given in Bureau of Standards *Technological Paper No. 111.*

When the structure in which the brick is used is located in a climate subject to freezing weather, the question of freezing and thawing becomes one of the most important factors to be considered in determining an allowable percentage of moisture in building brick.

A very interesting paper on freezing and thawing and the correlation of sodium sulfate treatment with actual freezing of building brick was presented to the 1919 meeting of the Society by Edward Orton, Jr.[1] Without going into this exhaustive report, it is of interest that Mr. Orton estimated that 4 sodium sulfate treatments were about equivalent to 25 actual freezings. He presented with this report a comparison between the grading of 37 groups of 5 bricks each, which in addition to the value of the tests illustrated forcibly the difference in strength values of the A.S.T.M. tests for brick of different clays. It is my suggestion that further tests should be made on brick by both the sodium sulfate and actual freezings in order to determine a more definite correlation between these two methods, always bearing in mind the actual conditions of use of the brick in construction. Due to the time and expense involved in making actual freezing and thawing tests, it may be advisable to use the sodium-sulfate tests for bricks of a certain percentage of absorption and actual freezing tests for the rest. Further tests will also help to determine the most important fact to consider in connection with absorption and that is whether the percentage of absorption is really of vital importance in making a classification of building brick.

It is of interest to note what has been done in testing other products to ascertain the effects of freezing and thawing. In a pamphlet on the "Strength, Absorption and Freezing Resistance of Hollow Building Tile" issued by the Bureau of Standards, Mr. H. D. Foster says that "an absorption of 16 per cent apparently marks a very definite line in the resistance to freezing and thawing. Tile of lower absorption showed little or no effects from 100 freezings and thawings." Due to the fact that brick is much thicker than hollow tile, it would seem that if the actual danger point for brick can be determined, it will be greatly in excess of 16 per cent and probably at a certain point be dependent on the modulus of rupture.

Regarding absorption in general, the possibility of varying percentages of moisture affecting actual construction work in any of the ways previously mentioned should be determined and then a recommendation made as to how much absorption should be allowed

[1] Edward Orton, Jr., "A study of the Proposed A.S.T.M. Tentative Specifications for Building Brick and a Correlation of Their Requirements with Sodium-Sulfate Treatment and actual Freezing." *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part I, p. 268 (1919).

for different uses. The Building Code of the City of Boston requires that the brick be tested for compressive strength dry and then saturated. The difference in strength, if any, between the dry and wet brick, is the determining factor in allowing the material to be used and not the percentage of absorption.

The Building Code Committee of the U. S. Department of Commerce in its tentative code for masonry walls recommends the Specifications of the A.S.T.M. with the radical departure, "That when the average compressive strength of brick grading soft by the absorption test is more than 3000 lb. per sq. in., the requirements as to absorption may be ignored." The A.S.T.M. gradings make it necessary to classify a brick by the results of one test rather than the combination of all tests and so when the percentage of absorption is higher than that allowed for a hard brick, the brick must be classified as medium or soft. It does not seem logical that the absorption should be made the deciding factor when the compressive and tensile strength of a brick may be very high if the absorption does not weaken the brick for practical purposes.

Bricks in actual use are subject to wetting by standing water in the case of foundations and by falling rain in the case of exterior walls. Tests should therefore be made to simulate these conditions as nearly as possible. In the present specifications of the Society, it is provided that the brick be boiled in water for five hours. In some other specifications, the method by which the absorption is determined, is without boiling.

Another point should be considered, namely, the rate of absorption. When the brick is used in the wall, it may, if used on a corner, have two surfaces exposed to the weather and the other four surfaces covered with mortar. The moisture which the brick will absorb will come either from the two exposed surfaces or from the mortar. The brick when boiled can liberate practically all the air which is held inside and which is displaced when the brick is thoroughly saturated. The brick when immersed can liberate eventually practically all of the air. Yet when it is built into the wall, it cannot liberate the air as readily amd it is doubtful if it would ever reach the maximum of absorption. The rate of absorption should indicate how easily the air would be driven out of the brick and would probably give some idea of what the actual absorption would be in the wall and conversely the evaporation which would dry out a wall after wetting.

The author suggests that a determination be made of the amount of time which it would require the maximum rainfall in any locality, driven by the average wind pressure in that locality, against walls

of given thicknesses constructed with good sound hard brick of that locality in diferent kinds of mortar, to penetrate such walls to the greatest saturation possible under the stated conditions. I should also want to know not so much that two bricks at the end of five hours each had, in a laboratory test, absorbed the same amount of water but which brick had absorbed the most water the quickest. If at the end of one hour or two hours, one of the bricks had become saturated but the other one had required the full five hours to reach its saturation point then I would have some index of the time required to penetrate a wall. But, this would have to be checked with the wall tests, where the mortar joints play such an important part.

There is another factor which seems to have great bearing on the interpretation of laboratory tests of brick in reference to absorption and freezing and thawing. Practically all tests that have been made heretofore have been made on new brick. There are, however, thousands of buildings in this country in which brick has been subjected to the rigours of a hundred and more severe winters and an equal number of torrid summers. By testing these brick, when they are available, a knowledge of how freezing and thawing has really affected brick might be obtained and this information used as another basis on which to interpret the results of laboratory tests for absorption.

What we all would like to see determined, if possible, is just what life or duration in certain localities may be considered as equivalent to five hours of boiling. Just what, in years of service, may be represented by 25 freezing and thawing tests? by 50 such? or by 100? or by 200? What matter it to us, or to the people who pay for and live in buildings, what the results might be of tests on the individual brick, for compression, for modulus of rupture, and for absorption if the tests did not indicate that the brick would give the desired service in the wall.

To these three requirements for individual brick we must add another that will give an index to durability under exposure in the wall—or establish one or two tests that will cover all.

The boiling tests and freezing and thawing tests made on a thoroughly saturated brick give a basis on which to calculate a laboratory strength for the brick. It is then necessary to interpolate from this laboratory analysis to arrive at a conclusion which will be substantiated in actual practice. Take as a well-known example, Independence Hall, Philadelphia. This building has been subjected to over a hundred and fifty winters of freezing and thawing and the bricks are apparently in perfect condition. It is not known how the

A.S.T.M. tests might have classified these brick, but the average laboratory tests might not have indicated this wonderful life and service rendered to date with apparently centuries yet to go.

Mortar and Bond:

The use of building brick which we are considering is in some form of construction as units bonded together by a mortar. As the mortar is a considerable percentage of the actual wall, the strength of the wall depends to a great extent upon the strength of the mortar. Tests have shown that in some instances a wall will develop a greater compressive strength than the strength of the mortar tested alone. Mortar is usually tested in cubical or cylindrical shapes but is not used in that form which accounts for some difference.

Since the strength of the brick is greater than the strength of the mortar, it is important to find the strength of the mortar as well as the strength of the brick. Some building codes have taken this into consideration as, for example, the code of the National Board of Fire Underwriters which has been adopted practically *in toto* by many cities. In this code, it is required that bricks be tested *flatwise* for compression. No modulus of rupture is specified. The brick must develop 3000 lb. per sq. in. compressive strength but when laid up in a wall, the load allowed is limited as follows:

Brickwork in Portland Cement Mortar.....	250 lb. per sq. in.
Brickwork in Natural Cement Mortar.....	208 "
Brickwork in Lime and Portland Cement Mortar.	208 "
Brickwork in Lime Mortar.....	111 "

It can be seen from this that while the brick must pass certain tests, the use of the brick is determined not by the strength of the brick itself, but by the mortar. Why should a brick have to be the same strength, where only a load of 111 lb. per sq. in. is allowed, as it would have to be when 250 lb. per sq. in. is allowed? If the brick would last as long and not deteriorate, it is only reasonable that a weaker brick would do for a lighter load or *per contra* that a stronger brick should be allowed a greater load. The only logical answer to this is that a brick of a certain compressive strength would normally be hard and durable. This again would vary with different clays.

Irrespective of any required compressive strength in the individual brick, it is evident that the mortar used is of great importance. Tests of brick piers built with different mortars were made at Stockholm by Kreuger and the results are interesting:

INFLUENCE OF THE STRENGTH OF MORTAR ON PIERS.

Pier	Compressive Strength of Bricks, lb. per sq. in.	Compressive Strength of Mortar, lb. per sq. in.	Mortar Mixture	Compressive Strength of Piers, lb. per sq. in.
No. 1	4040	0	Dry sand	740
No. 2	4040	38	1 lime: 3 sand	740
No. 3	4040	355	2 lime: 1 cement: 9 sand	1420
No. 4	4040	695	1 lime: 1 cement: 6 sand	1840
No. 5	4040	1280	1 lime: 2 cement: 9 sand	1700
No. 6	4040	1640	2 lime: 1 cement: 7 sand	1930
No. 7	4040	2620	1 cement: 3 sand	1980

Tests on these bricks were not made in the same way as they are made in the United States but they give some idea of the effect of mortar on the ultimate strength of a pier. The modulus of rupture is not given and cannot therefore be considered in making deductions.

Besides the mortar, the bond or method of laying the brick in the wall makes a considerable difference in the strength of the wall as was shown by the tests on piers made at the Watertown Arsenal.

Method of Laying Brick	Percentage Gain in Strength by Various Methods of Laying Brick
Bricks Flat:	
Joints broken every course	0
Joints broken every sixth course	7.2
Bricks on Edge:	
Joints broken every course	43.6
Joints broken every third course	57.1

As far as the real safety for all ordinary construction is concerned, it would seem as important to study the adhesion of the mortar to the bricks and the method of laying the bricks in the wall as to determine the compressive strength of the wall. The old method of stiffening walls was to make them thicker. This method produced results but was not very scientific or economic. With the more modern custom of using thin walls, it seems very important to determine which bond and which mortar develop the maximum strength.

In connection with absorption, it was mentioned that the percentage of absorption would influence the adhesion between the mortar and the brick. This difference also should be determined.

Tensile Strength:

Tests on the tensile strength of a brick are not yet available for publication but it seems very probable that the tensile strength has a bearing not only on the compressive strength but on the modulus of rupture which is more important. This subject would be an interesting one to study because of the numerous kinds of clay and several

ways of making brick all of which might affect the tensile strength. Stiff-mud bricks may be end cut or side cut and since the augering operation is the same for all cuts, the "grain" of the brick would be different in each case, varying with the clay or shale.

This subject of cutting the brick in manufacture should also be studied to determine the effect of the adhesion of the brick with the mortar and consequently the effect on the strength of the wall.

Size of Building Brick and Tolerances:

The Standard Specifications for Building Brick of the Society now state that "The standard size of building brick shall be $2\frac{1}{4}$ by $3\frac{3}{4}$ by 8 in." This size was determined upon after mature consideration and several revisions and represents not only the standard of this Society but also the accepted size of all brick manufacturers associations, with the exception of a $\frac{1}{8}$-in. increase in width for smooth face brick. However, the above figures were accepted by all of these associations, Governmental Departments and others in attendance at a conference in 1923 at the Division of Simplified Practice of the U. S. Department of Commerce with the important addition that this size was considered as approximate only and represents the standard size to be aimed at by the manufacturer for the average of his entire production. Such a statement, which has been promulgated by the Division of Simplified Practice, should undoubtedly be immediately incorporated in the A.S.T.M. specifications.

In Conclusion.—It is, of course, the desire of the American Society for Testing Materials and of its committees to have its specifications and standards, which represent years of the most exacting laboratory research, many tests, and exhaustive study and deliberation, referred to and used wherever and whenever possible. I trust, therefore, that fullest encouragement and cooperation will be offered to all proper and authoritative agencies in the conducting of tests on new and old brickwork in walls and piers of all types and under all conditions sufficient to reach definite conclusions for specificational requirements covering all grades of individual brick that will insure their appropriate and economic use according to structural intent or climatic exposure.

It will be a large undertaking to get the right kind and amount of data necessary, the scope of which is merely indicated in this paper. But it can be done.

DISCUSSION

Mr. Lawson.

MR. T. R. LAWSON.[1]—I think I represent the sentiment of all the members of Committee C-3 on Brick when I say that this paper is a very timely one. It has brought up a great many points which have been thoroughly discussed by committees and sub-committees and there has been a feeling that the Society at large should know a little more of our activities. This paper sets the situation clearly before the Society.

Some of us do not entirely agree with everything that Mr. Boyd has brought out and which has been done largely for the purpose of giving you something to shoot at, for if you have nothing to discuss, there will be no discussion. I am not sure that Mr. Boyd is entirely committed to everything he has said, but at the same time we are in the position of wanting to know what to do and how to do it, so that we can best serve the public in this matter.

I am only going to offer one or two thoughts that have occurred to me; one in particular has to do with item four of the Synopsis where the suggestion is made that we "classify the brick according to the two highest tests rather than the one lowest, with perhaps limits on that property which is to be least considered, rather than waiving them as is now provided for by at least one national authority." There would be considerable danger in this, as in some instances it may amount to gaging the strength of a chain by its strongest rather than by its weakest link, and it should be handled with a considerable degree of care.

The point which has brought this up is the fact that in some instances the crushing resistance, the transverse strength and the absorption of the brick are so widely at variance that it has called for the abandonment of the absorption when the crushing strength reaches a certain limit. This might defeat the entire purpose of the specifications and if such an arrangement were made that any one test is to be abandoned, considerable care ought to be exercised as to just where that brick is going to be used, as it may be a vital factor.

There has been some misunderstanding as to the purpose of the absorption test. Some have felt that it has nothing to do with anything but the ability of the brick to resist frost action. This is not entirely so. I think Mr. Boyd has pointed out very clearly that the

[1] Professor of Rational and Technical Mechanics, Rensselaer Polytechnic Institute, Troy, N. Y.

ability of a brick to withstand a load may be affected very materially by the amount of water that it will absorb. We have not a great deal of data on the crushing resistance of bricks that are dry as compared with the same bricks that have a maximum amount of water in them, and data on this would be very acceptable and I believe would be of considerable value. Mr. Lawson.

Another test has been suggested concerning which we have no data and that is the question of surface hardness and how it shall be determined. Another test that needs a considerable amount of experimentation is that of the artificial methods of conducting the freezing test. We have no specification for it at present, but we believe that we should have more data than we now have on the subject of the alternate freezing and thawing of brick. In order for this test to be effective, it means that we must have a great number of freezings. It is not sufficient to have a brick alternately frozen and thawed for a half dozen times, the process should be continued until disintegration occurs, even if it takes 150 or 200 freezings. Obviously, the expense and time involved in performing a test of this character is so great that it is impractical. We are therefore in need of a substitute test which will develop this particular property.

MR. W. C. KOCH.[1]—I should like briefly to point out one thing that is right in line with what Mr. Lawson has said. There is need of invention in this matter of tests of brick. He has intimated that in some ways we have not progressed far enough to do a good job in the testing of brick and I think that the following is one of the reasons for that: As far as the freezing and thawing and disintegrating of brick is concerned, there is not merely one problem; there are two, and those two problems are just as distinct from each other as the testing of the brick and the testing of a boiler. An underburned brick will disintegrate under a sufficient number of freezings and thawings. It will disintegrate particle for particle. It will appear at the end of a period of disintegration as if it had actually been sand-papered. That is one type of frost disintegration which requires a certain test to determine in advance. Mr. Koch.

The other type of frost disintegration has nothing to do with that; it is the spalling type that occurs through an entirely different cause. The first type occurs because of the lack of bond within the brick, a bond which has never been set up there since the brick is not sufficiently burned. The other type has nothing to do with the interior bond of the material. It has to do with the structural labyrinth of capillaries and results in actual partition.

[1] Vice-President and General Manager, Twin City Brick Co., St. Paul, Minn.

Mr. Koch. The result is spalling and flaking and that requires an entirely different test from the underburned disintegration problem. I think that invention will be required to separate those two problems and to devise a good test for each of them.

Mr. Woolson. MR. IRA H. WOOLSON.[1]—I think that the Society is certainly to be congratulated on having a paper presented to it by an architect on such a time-worn subject as brick—and such an excellent paper. Unfortunately, we get very few papers from architects and very little cooperative effort on the subject of testing of materials and there is certainly no class of users or fabricators who should be better posted or who ought to take more interest in the subject of quality of materials than the architests.

There is one thought Mr. Boyd brought out concerning which I should like to ask him just what his practical solution of it would be. He questions the advisability, or the desirability perhaps, of requiring brick of the same strength in walls that are laid up with different mortars. He calls attention to the fact that the working stresses of walls as a rule, are predicated upon the strength of the mortar and then says that it would seem illogical to have the brick all of the same strength.

My conception of that would be that a brickyard would manufacture its brick, and the average run of that brick as classified according to our standard specifications would throw them into probably two, possibly three classes. If a man were putting up a building, would it be worth his while to attempt to buy the brick of different strengths for different walls? Perhaps that is not the thing he was aiming at, but he spoke of walls in general and I assume he would include an outside wall. Was his thought that because a building was low, or the floor-carrying capacities of the occupancy was low, that therefore a low-grade brick might be used in that building? It seems to me that would not be practical in any particular locality.

Mr. Boyd. MR. D. K. BOYD.—I will attempt to reply to Mr. Woolson first. He almost answered the question for me by stating that it was a matter of differential or preference in the use or the allocation of the brick in a building. We must assume that architects, engineers and builders should appropriately use all good products. We could not say that bricks which were soft and yet necessarily part of the production of a kiln must be thrown away; otherwise the remaining bricks would be prohibitive in cost.

Take the City of Philadelphia, for instance. It is common practice there to build the dividing walls, which are known as party walls,

[1] Consulting Engineer, National Board of Fire Underwriters, New York City.

between row houses (and unfortunately we have too many row houses in Philadelphia) of "salmon" brick. An operative builder who orders brick to-day will invariably order certain kinds for different uses. Formerly they used to have what they called a front brick and a back front brick and a filler, making three kinds. They used the pressed brick on the street front of the house, the back front exposed to the yards was faced with "stretcher" brick and the interior or fillers of those walls and the party walls were built of the softer bricks. Mr. Boyd.

Now, of course, the manufacturers may not have had enough of the soft or "salmon" brick to fill the order and then they would furnish their regular hard-burned brick. So it seems to me it is a matter of selection and appropriate use.

Mr. Koch spoke of the freezing and thawing, as did Mr. Lawson. I think the answer there lies in the fact that there are throughout this country, and all the European countries, buildings of brick that are not only one but several centuries old. They are in northern climates where the bricks are subjected to very cold weather, to rain and freezing and then a thaw followed by rain and freezing. We have in our country, as for instance Independence Hall in Philadelphia and Faneuil Hall in Boston and other buildings in our eastern seaboard cities, many which are 150 years old or thereabouts and which have had their freezing and thawing going on all winter and which then are subjected to a torrid summer. They have succeeded in withstanding that for we will say 150 years and have had perhaps ten freezings and thawings during a winter, and in the vast majority of cases they are in fine condition and performing their function to-day and preserving us buildings which are monuments not only historically but architecturally.

In regard to Mr. Lawson's first statement about basing the classification of a brick on compliance with two requirements instead of three, I would merely call attention to the fact that I suggest the possibility of limiting the third classification, whichever one it might be, and not eliminating it as has already been done in at least one instance referred to.

With respect to the remarks of Mr. Woolson, I feel that it is more advisable to limit the one classification which we would least consider rather than to ignore it. Instead of saying that if the brick has a certain compressive strength and a certain modulus of rupture, its absorption may be ignored, I would suggest saying that the absorption may go to but not exceed 25 per cent or 30 per cent or put some outside limit on it.

Mr. Randall. MR. T. A. RANDALL.[1]—The texture of the brick is an important element which makes it almost immune or proof against time and weather exposure. That is why the brick made in the earlier days, to which some of the speakers have referred, have proven indestructible though they may not have been so well made as those of to-day. They are not affected by freezing and thawing because the mass of material is so minutely or intimatly mixed that it is proof against the ravages of time. As an illustration, some thirty odd years ago I built a home and roofed it with clay tile. In the course of construction it developed that the tile were very absorptive, so much so, that I became uneasy lest they would not stand our severe winters in Indiana. The manufacturers of the tile assured me there was no danger from that source for though the tile would absorb water freely, the texture was so fine that freezing or thawing would have little or no effect. That proved true, the roof is as good to-day apparently as when first put on and there has been no leakage.

The modern methods of manufacture are quite different from the older methods. Shale as well as common clay is not infrequently used in the manufacture of brick, and improved methods of burning enable the average manufacturer to produce a brick which is superior in every way to those made by hand in the early days.

In the meeting of Committee C-3 yesterday the question was raised why so many manufacturers of brick in this country pay so little attention to specifications and standardization of brick. The answer to that is largely in the fact that the indestructible quality of brick is such that they exceed all ordinary factors of safety to such an extent there is no apparent need for specific standards. This is particularly true in localities where brick have been made and used for years and years, and have proven adequate in every way. You know as a rule the brick industry is a local industry. In metropolitan districts the leading manufactors are alive to the importance of proper standardization and I am sure the work of our Committee on Brick will prove helpful to the industry and to the builders.

Mr. Lawson. MR. LAWSON.—Mr. Ingberg, in presenting the report of Committee C-10 on Hollow Masonry Building Units mentioned the fact that many of the tile which with a small number of freezings did not show evidence of disintegration did commence to go to pieces between fifty and sixty freezings. I am wondering whether or not he has made similar tests on brick and whether the same relation was shown.

[1] Secretary, National Brick Manufacturers' Association, Indianapolis, Ind.

MR. S. H. INGBERG.[1]—We tested brick along with the tile. We did not have exactly similar clays in brick and tile but in the brick we had a range of hardness all the way from a shale with an absorption of only a few per cent to a surface clay with an absorption of over twenty per cent, all being well burned. There was a difference in behavior there and I think to get the same effects you would have to continue the tests on brick longer. In fact, we ran most of the brick to 300 freezings and on some of them the effects were only minor. The clays were not strictly comparable, so perhaps any conclusions that may be drawn are not accurate so far as relative resistance due to the form and size of the unit is concerned. Mr. Ingberg.

MR. WOOLSON.—I should like to ask Mr. Ingberg if during their experiments with freezing and thawing they have tried the sulfate process at all? It is an old method that was suggested many years ago when we did not have facilities for quick artificial freezing. Mr. Woolson.

MR. INGBERG.—The Bureau of Standards has made tests on clay materials with the sulfate treatment. I am not particularly familiar with what has been done on brick. I know for clay materials, generally, we are beginning to almost give up the sodium sulfate test. I am not certain that I can indicate exactly why. The sulfate treatment puts a chemical into the structure that expands on crystallizing. The only effect you get is a bursting pressure from the expansion. The freezing treatment involves this in part, although with water instead of a chemical. The pressures of crystallization of the two are necessarily very different. Again, in freezing we have quite sudden temperature changes when the brick is immersed in the freezing chamber or is being thawed which produce stresses in the outer layers of the brick that do not obtain in the sodium sulfate test. The disintegration in alternate freezing and thawing may be due as much to stresses induced by the temperature changes as to the bursting pressure of the ice. Mr. Ingberg.

It is apparent that the alternate freezing and thawing test, using water, is a reliable test to determine weathering properties. The only point that is indefinite at the present time is the number of alternations that should be required in order that a clay product may be considered to have satisfactory properties and there is a way now by which it appears possible that this can be determined, at least for brick. We can get brick from buildings that have withstood weathering for 100 or more years. If we put such brick into the freezing chamber, determine its absorption and other properties, determine the point where disintegration begins, then we have a check on the

[1] Physicist, U. S. Bureau of Standards.

Mr. Ingberg. laboratory test. Some work of that kind is being planned at the Bureau. Right parallel with them can be put the brick of to-day and we can check the effects on them against those on the 100-year-old brick.

Mr. Carver. MR. WILLIAM CARVER.[1]—I wonder if I might offer the thought that an absorption test on brick is an inconclusive test and shows absolutely nothing unless we take into account the degree of hardness to which the brick has been burned. There are instances where an underburned brick might have low absorption, but such brick would in time disintegrate when exposed to the weather, whereas a well-burned brick, even though it has high absorption, stands unharmed for centuries. We must realize that the requirements that have been formulated to date for the testing of burned clay products, as Mr. Boyd has so ably brought out, do not give the kind of information that the user of these products should have.

Mr. Viens. MR. E. VIENS.[2]—I have had occasion to test stones for their ability to withstand the action of frost when wet or saturated with water, and found that neither the percentage of pore space nor the ratio of absorption is a criterion as to the frost resisting power of the stone. The best guide, to my mind, is the relation of the character of the pore spaces, that is, the coarse and the fine, which may be defined as capillaries and sub-capillaries. A stone of high absorption may withstand alternate freezings and thawings when wet or saturated better than one of low absorption, depending on the ratio which exists between the fine pore spaces (sub-capillaries) and the total pore space, which is known as the coefficient of saturation. If we determine this ratio, then we have obtained a factor which will indicate the ability of the stone to withstand the action of frost. It is possible that this method of testing stone could be applied to brick and other clay products in finding out their ability to withstand frost action.

Mr. Talbot. MR. A. N. TALBOT.[3]—Relative to this last discussion, I should like to say a word concerning the absorption test for brick. It seems to me we must accept as true that the absorption test of itself may not tell much about the quality of brick. Years ago I made some study on this with relation to paving brick and somewhat with respect to building brick. There is a great difference in the rate of absorption. There is a great difference, of course, in the amount of absorption.

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[2] Director, Laboratory for Testing Materials, Department of Public Works, Ottawa, Ont., Canada.

[3] Professor of Municipal and Sanitary Engineering, and In Charge of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

Mr. Talbot.

There is a great difference in the relation between the large spaces and the small spaces of the voids in the brick. I have seen brick having twenty or twenty-five per cent absorption that surely was a most durable brick. I have seen other brick with two or three per cent absorption that would go to pieces very readily.

Twenty-five years ago I suggested for paving brick a relative relation; that is, that for a particular brick there should be a limit made; let the manufacturer present the grades that he thought he could furnish and from those the quality was determined and later deliveries judged.

It seems to me that there is yet to be found a test on this matter of weathering. I do not agree because a brick from a certain location has lasted 100 years that a brick that is made to-day in that same place will last the same time. Changes are being made, changes in processes, changes in materials, changes in workmanship. Our specifications should have something which refers to the quality of the brick which is being put out and bought by the purchaser.

I should like to say a word concerning other tests that were discussed. I think all will agree that the making of tests and specifications for brick is one of the most difficult of all materials. There is such a variation in processes, in materials, and in the properties of the product over the country. So far as compressive strength is concerned, after a certain value for any given purpose is reached, the additional strength of itself is probably of little value. It is well known, and has been for many years, that the strength of walls and the strength of columns is due not so much to the compressive strength—very seldom is the load such as to crush the brick—but rather to the cross-breaking strength, the transverse strength.

That indicates that the cross-breaking strength has more value than the compressive strength. However, transverse tests are uneven; you must expect great variation in result; variation in the product itself, and probably a variation in the way in which the test load will be applied in one specimen and in another; so that a large number of specimens must be required. For the purpose of strength of walls, the transverse test does give more information concerning the brick than the compressive test.

I am very glad to hear Mr. Ingberg make his remark concerning the value of the sulfate test as an accelerated test. In the Committee on Drain Tile, with which I was connected some years ago, a great deal of discussion was given to the matter of an accelerated test and I believe the conclusion of all was that no accelerated test had as yet been found that would be serviceable and reliable. The freezing

Mr. Talbot. of water and the weathering in our climate is quite different in its action from that of this chemical.

I believe that the matter of determining the texture and the quality of brick other than by strength is something to which the committee may well give its attention.

Mr. Boyd. MR. BOYD.—Might I say, if it will be of interest to Mr. Ingberg, that I would be very glad to furnish him with several bricks, which we might very well term pedigreed bricks. They are from St. John's Church in Philadelphia, which has been standing, until within the last few months, for 117 years.

Might I also express my appreciation of the discussions presented. The points brought out by the various speakers are most interesting and the discussion should be helpful to the Committee on Brick.

A PRACTICAL METHOD FOR DETERMINING THE RELATIVE STABILITY OF FINE-AGGREGATE ASPHALT PAVING MIXTURES

By Prévost Hubbard[1] and F. C. Field[2]

Synopsis

The paper describes a laboratory test for determining the stability, or resistance to displacement, of asphalt paving mixtures, primarily of the sheet asphalt type. The test is being used to study and improve existing paving mixtures and specifications for such mixtures. The apparatus and method are described in detail and certain results are included to illustrate the usefulness of the test.

Briefly described, the test consists in forming a briquette from the hot paving mixture, allowing it to cool and age for a short period, then warming it to 140° F., placing it in a testing mold and measuring the maximum load required to force the mixture through a circular orifice. This load is recorded as the stability value of the mixture.

The test has been found to be well adapted to differentiating between various combinations of paving asphalt and mineral aggregate and it is believed that results obtained with the fine-aggregate mixtures will, when properly interpreted, increase our knowledge regarding the relative stability of coarse-aggregate paving mixtures.

It is shown that the addition of mineral filler up to 25 per cent materially increases the stability value of a natural sheet asphalt sand mixed with asphalt. That all mineral fillers are not alike in their stabilizing value is illustrated by the fact that, upon an equivalent weight basis, hydrated lime was found to be more effective than a commercial limestone filler. This bears out the results of earlier investigations of the stability of mineral aggregates mixed with castor oil and tested at normal temperature.

In the field of bituminous paving the necessity of laying paving mixtures of a high degree of stability or resistance to displacement has become a very important matter in order that such paving mixtures may successfully resist the stresses to which they are subjected by modern traffic. No method of ascertaining the relative stability of different mixtures except by service results over an extended period of time has as yet come into general use, although during the past few years a number of investigators have been at work upon this problem with the idea of developing a reliable laboratory test for the

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purpose. The very general interest in this subject is evidenced by the fact that three papers upon stability of bituminous paving mixtures have been scheduled for this meeting.

The purpose of this paper is to describe a test which has been devised by the authors for studying fine-aggregate asphalt paving mixtures, primarily of the sheet asphalt type, and to present a few test results to illustrate the adaptability of the test for the purpose intended. While in principle the test is a simple one, a great deal of work has been required to bring it to its present degree of accuracy and some improvements in detail of operation yet remain to be developed. It is believed, however, that in its present stage this test is of very practical value and gives promise not only of materially increasing our knowledge of the fundamentals governing the stability of fine-aggregate asphalt paving mixtures but of the coarse-aggregate mixtures as well, particularly those containing an appreciable proportion of bituminous mortar.

The principle of the test is based upon the assumption that displacement of a paving mixture under traffic, tending to the formation of waves or bumps, is mainly due to shearing stresses set up within the mixture by the movement over its surface of heavily loaded wheels; also that these stresses cause the greatest amount of displacement when the mixture is at its maximum summer temperature. Briefly described, a test briquette is first formed of the paving composition, which is mixed and compressed at the same temperature used in laying the pavement. The briquette is then brought to a temperature approximating the pavement temperature in hot weather, placed in a testing mold and the maximum load required to force the mixture through a standard orifice is recorded as a measure of its resistance to displacement.

The principal parts of the briquette molding and testing apparatus are shown in Fig. 1 and consist of a forming mold, bottom plate, plunger and compression plate, testing mold, testing ring and testing ring clamp.

The forming mold is a steel tube $4\frac{3}{4}$ in. long with an inside diameter machined to exactly 2 in. When forming the test specimen, one end is placed upon the steel bottom plate which is about $3\frac{1}{2}$ in. square and $\frac{1}{4}$ in. thick. Three forming molds and bottom plates are used in making a batch of three test specimens.

The plunger is made from a steel rod first turned to a diameter of 1.98 in. It is then milled so as to produce six longitudinal ribs each about $\frac{1}{4}$ in. in width measured on the original circumference, the depth of milling being $\frac{1}{16}$ in. To the bottom of the plunger is fastened

the compression plate, which consists of a hard steel disk 1.998 in. in diameter at the bottom and slightly tapered to the diameter of the plunger throughout its thickness of $\frac{1}{8}$ in. The compression plate is attached to the bottom of the plunger by means of three screws, the

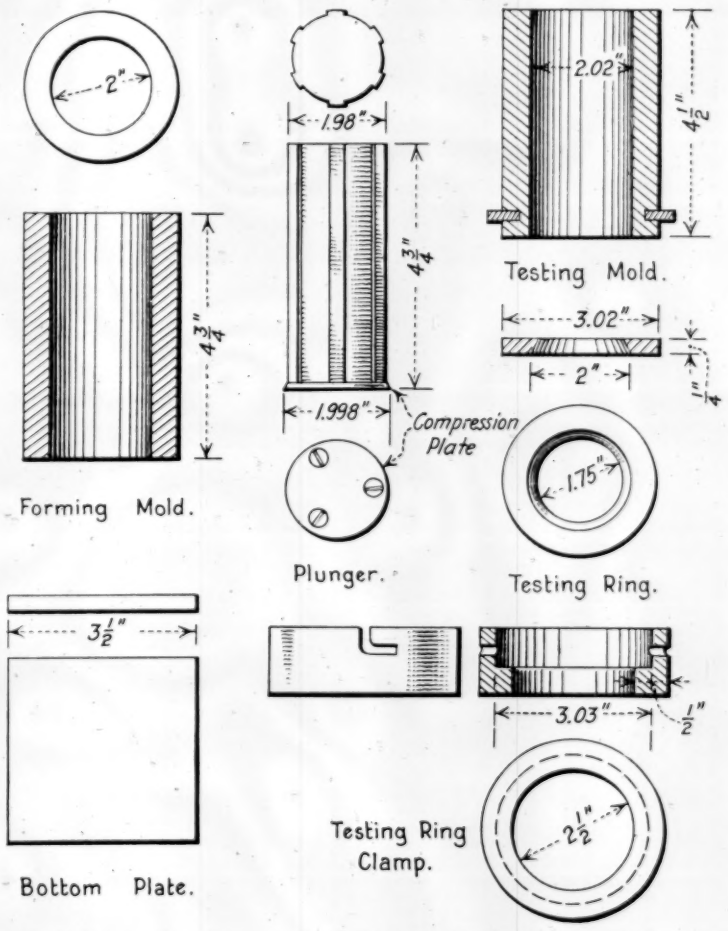

FIG. 1.—Details of Forming and Testing Molds, Plunger, Testing Ring and Ring Clamp.

heads of which are set flush with the face of the disk. The plunger and compression plate are designed to reduce wall friction as much as possible when forming and testing the briquettes.

The testing mold is similar to the forming mold but is $4\frac{1}{2}$ in. long and turned to an inside diameter of 2.02 in. This increase of

0.02 in. in diameter makes it possible to easily insert the pre-molded test specimen just prior to test and practically eliminates wall friction from the specimen. The testing mold has an outside diameter of 3 in. and carries two projecting pins placed near the bottom and opposite to each other.

The testing ring is made from a piece of hard steel $\frac{1}{4}$ in. thick and has an outside diameter of 3.02 in. The circular opening which forms the orifice, through which a portion of the briquette is forced

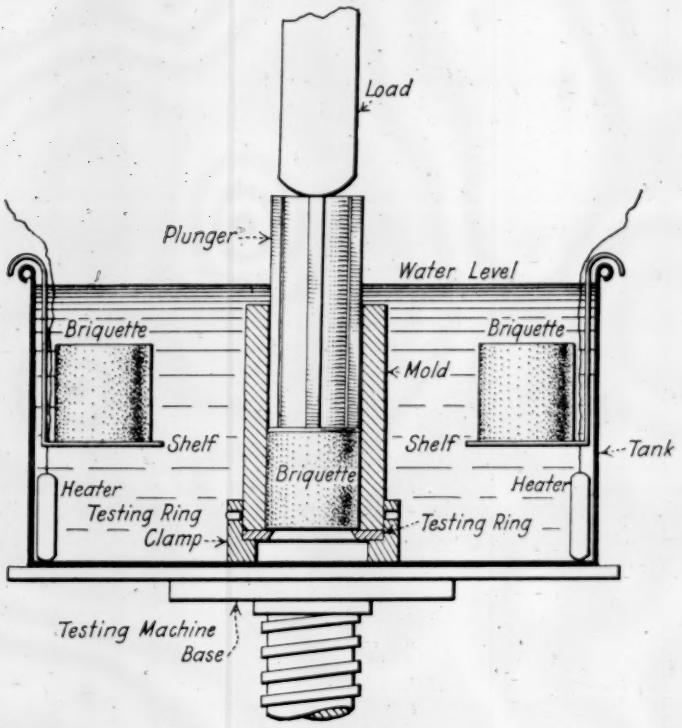

FIG. 2.—Stability Test Assembly.

during test, has a diameter of $1\frac{3}{4}$ in. This opening is tapered to a diameter of 2 in. at the under side of the ring. During a test the testing ring is held tightly against the bottom of the testing mold by means of the testing ring clamp.

The testing ring clamp consists of a steel collar with a maximum inside diameter of 3.03 in. which permits the easy insertion of the bottom portion of the testing mold. Locking slots are cut in the wall to admit the pins on the testing mold and are so shaped that a slight turn of the clamp locks the testing ring firmly against the bottom of

the mold. The lower portion of the testing ring clamp has an inside diameter of $2\frac{1}{2}$ in. extending for a distance of $\frac{1}{2}$ in. A shoulder is thus formed $\frac{1}{2}$ in. above the bottom upon which is first placed the testing ring when assembling the apparatus. The space below the testing ring is sufficient to hold the test mixture which is forced out of the mold during test.

As specimens are tested under water at an elevated temperature, in addition to the above described apparatus there is required a water bath consisting of a heavy galvanized sheet-iron tank 12 in. square and

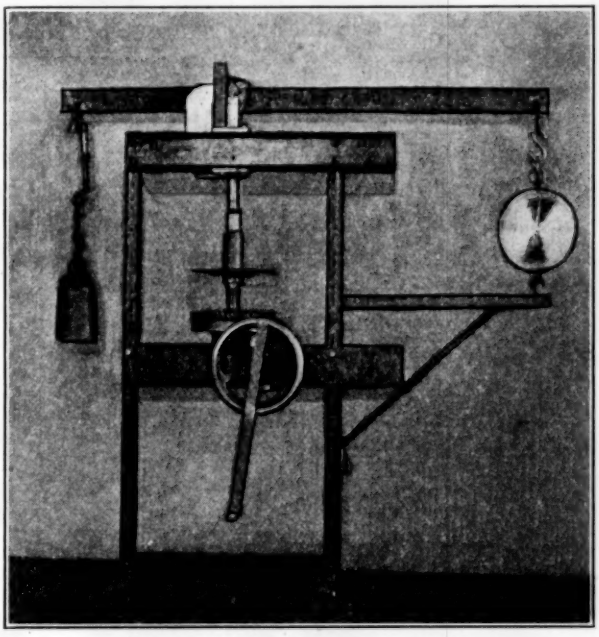

FIG. 3.—Testing Machine with Stability Test Apparatus in Place.

6 in. deep. A circumferential metal shelf $2\frac{1}{2}$ in. wide is suspended from the rim of the tank $2\frac{1}{2}$ in. above the bottom. This shelf is used to hold test specimens while being brought to the desired temperature. The tank is equipped with two electric heating elements controlled by a rheostat so as to maintain the water at the desired temperature. This equipment is illustrated in Fig. 2.

The testing machine for applying the load was especially designed for the purpose by F. Reichmann of John Chatillon and Sons. It was constructed for research work only, using for framework a Manley screw press which the authors had employed in earlier investigations. Its capacity is considerably greater than will ordinarily be required

for routine tests and as soon as sufficient data are obtained it is proposed to construct a simpler and more compact machine for general laboratory use along lines devised by Mr. Reichmann.

A photograph of the machine so far used is shown in Fig. 3, in which it is seen that the base, upon which rests the testing mold and plunger, may be raised or lowered by means of gears connected to a hand wheel carrying an extension arm. As the base is raised the plunger is forced against the ball-bearing end of a steel rod to the top of which is attached one of the main knife-edge bearings. This bearing is held at all times in contact with the knife edge by springs attached to the rod and overhead frame. By means of two additional knife edges the load applied to the lower end of the rod is transmitted to the horizontal counterpoised lever, to the long end of which is attached a 400-lb. spring balance with a dial graduated to 1-lb. readings. The lower end of the balance is attached to an extension rod fastened to the framework of the machine. The ratio of transmission of load is 25.86 to 1, so that the working capacity of the balance is increased to somewhat over 10,000 lb. The graduations on the dial are sufficiently spaced to permit of readings to $\frac{1}{4}$ lb., which represents an actual load application of 6.5 lb. In addition to the regular pointer the dial is equipped with a loose push pointer which remains at the maximum load registered by the test after the load has fallen off. This accessory is very necessary in order to obtain exact readings. The capacity of the machine may be increased to 20,000 lb. by substituting an 800-lb. spring balance for the one described.

Experience has indicated that carefully standardized control of the method of preparing test specimens is essential to obtaining close check results. The method at present used will therefore be described in considerable detail. Sheet asphalt paving mixtures are composed of graded sand, mineral filler and asphalt cement and it is not only important to proportion and combine these constituents accurately but to take precautions that their original characteristics undergo no unusual change during manipulation. Control of sand grading and also heat treatment before and during the mixing and molding operations are of great importance. Test specimens are prepared in batches of three each and three forming molds are used to a batch.

The sand for each batch should be representative of the product to be studied and should be carefully sampled by methods which are too well known to require description in this paper. When it is wished to study some particular grading, it is desirable to separate the sand into groups of different sized particles by means of testing sieves and to proportion these groups by weight for each batch pre-

pared. The total weight of a batch is about 660 g., so that the required amount of each constituent will vary with the proportions in which they are to be combined. The dry sand is first weighed out and mixed in a small bread pan with the proper amount of mineral filler. A typical batch might consist of 510 g. of sand and 90 g. of filler. This mixture is placed in an electric oven and heated to 400° F. About 100 g. of asphalt are also heated to 310° F. ±10° F. in a small deep aluminum cup just prior to its combination with the mineral aggregate, the period of heating not exceeding 20 minutes. The hot mineral aggregate is next transferred to a two-quart enameled-ware bucket with rounded bottom, the tare of which has been previously ascertained, and the correct amount of asphalt weighed in. Bucket and contents are then placed upon a sand bath maintained at about 400° F. and the constituents thoroughly mixed for four minutes with a $1\frac{1}{4}$-in. putty knife. Three forming molds and bottom plates together with the plunger which have been heated to 350° F. are then placed upon the work bench and into each mold is quickly placed about 190 g. of the mixture which is weighed out of the mixing bucket. As soon as each charge is placed in the mold it is lightly compressed by hand application of the plunger.

The specimens are then compressed by inserting the plunger in the mold first filled and placing this mold with bottom plate on the base of the compression machine. A load of 9425 lb. (3000 lb. per sq. in.) is then rapidly applied and after the dial registers this load the specimen is allowed to remain under pressure for exactly one minute without readjustment of the pressure in case it falls off. The load is then released and the first mold removed. Exactly the same procedure is followed with the other two molds in the order in which they were filled. The total time consumed between weighing the hot asphalt into the mix and completing compression of the third specimen should be from 12 to 14 minutes. Test results so far obtained have seemed to indicate the desirability of confining this operation to a period of not over 15 minutes. After the three specimens have been compressed they are at once removed from the molds and allowed to remain in air at room temperature from 18 to 20 hours before testing. In order to remove the briquette the mold may be placed on a shallow ring in the testing machine and pressure applied to the plunger until movement starts, after which hand pressure may be used to complete the operation.

Each briquette will be 2 in. in diameter and a little less than 2 in. high. After cooling and just before testing, its specific gravity may be quickly determined by suspending it on a 200-g. spring scale, grad-

uated to 1-g. readings, such as used for sand grading determinations, and weighing it in air and in water. The percentage of voids may then be calculated from the known proportion of the various constituents, provided the specific gravity of each constituent has been ascertained.

Owing to the fact that the density of the test specimens is never absolutely uniform throughout their depth, they should be marked on top as they were originally compressed and the top side should be placed up in the testing mold. Prior to testing, the testing mold apparatus should be assembled by clamping the testing ring tightly against the lower end of the mold with the testing ring clamp. This apparatus together with the test specimens are then placed in the water bath for one hour, being maintained at a temperature of 140° F. by means of the electric heater and the addition of heated water when necessary.

Each specimen is tested by placing it, original bottom end down, in the testing mold and inserting the plunger. The complete assembly ready for test is illustrated in Fig. 2. Load is applied by operating the hand wheel at the rate of one revolution per second which raises the base of the machine about 1 in. in 25 seconds. Revolutions of the hand wheel are timed with a metronome set to beat at half-second intervals. As the test mixture is loaded it commences to exude through the orifice in the testing ring and the dial pointer rises quite rapidly to a maximum after which, if the operation is continued, it fluctuates below the maximum as flow of the mixture progresses. The maximum load registered by the push pointer is recorded as the stability value of the specimen. After testing, the mixture is removed from the mold, which is cleaned before inserting the next specimen. The test itself appears to be quite delicate and between three test specimens of the same batch there will usually be a variation of less than 10 per cent. The average of three tests is reported as the stability value of the mixture.

Among the variables affecting this test, those of time, temperature and pressure are of course important. In so far as the test itself is concerned it is believed that these variables have been fairly well standardized, but occasional wide variations between results obtained on different batches of supposedly the same mixture have led the authors to believe that there is room for possible improvement in the method of preparing the mixture. It is also possible that a briquette of different height will be better adapted to the test, as at present conducted. The time element is an important consideration as applied to method of heating and preparation of the mixture, maximum load-

ing of the briquette and rate of application of the load during testing. Temperature is an important consideration in so far as heating, mixing and compressing the mixture is concerned and also of course in connection with the specimen during test. Pressure is important in connection with the molding of the specimen and will be inaccurately recorded if wall friction is not eliminated. It is believed, however, that variations due to wall friction have been reduced to a minimum by the

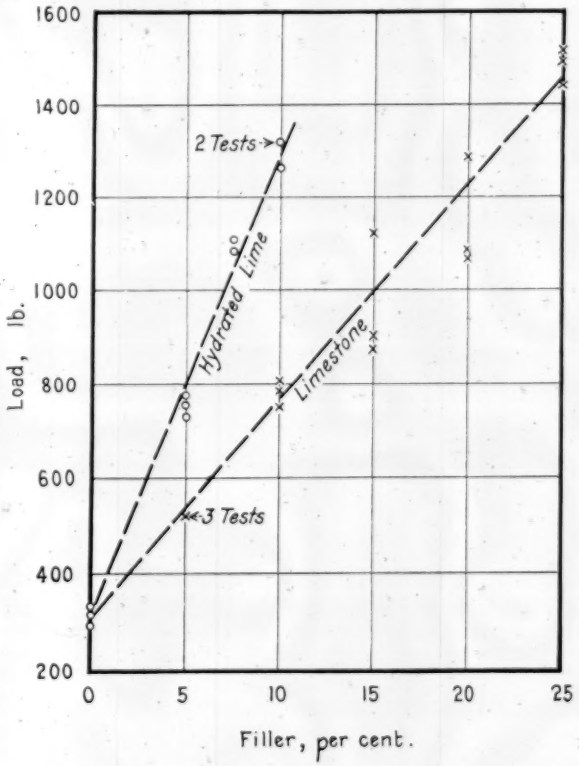

FIG. 4.—Effect of Filler on Stability of Natural Sand Mixtures with 10 per cent of Added Asphalt.

construction of the plunger and making the briquette slightly smaller in diameter than the testing mold. This smaller diameter apparently allows a thin water wall to form between the briquette and interior of the mold, which wall is maintained throughout the test. At any rate, little or no lateral distortion of the briquette takes place and after the extruded portion has been removed the remainder of the briquette slides out of the mold as easily as it went in. Density of the test specimen also has an effect upon results obtained and for a given mix-

ture this is largely controlled by the maximum pressure used to form the briquette. Indications point, however, to the danger of applying a load greater than 3000 lb. per sq. in. owing to crushing of a portion of the mineral particles which commences at about that pressure.

The number of details affecting any mechanical test of bituminous paving mixtures is legion and any attempt to discuss them fully would extend this paper to an unwarranted length. It is felt, however, that these details are gradually being mastered and that with slight modifications not affecting the principles involved in the test itself, its method of operation may be improved.

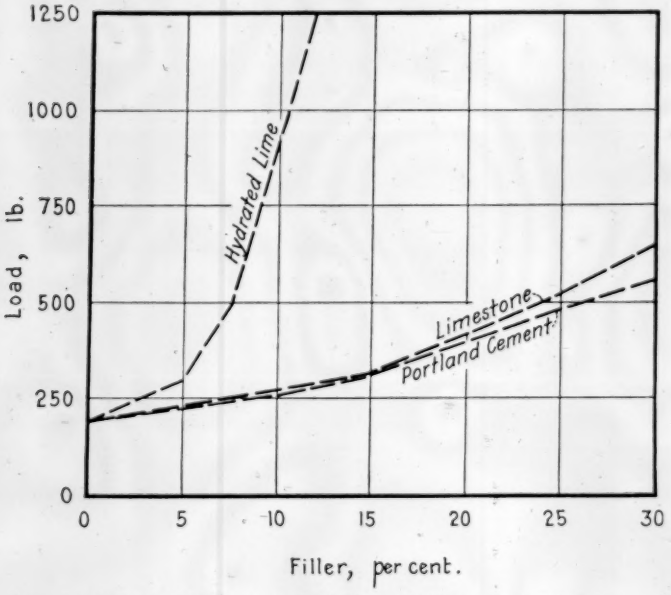

FIG. 5.—Effect of Filler on Stability of Natural Sand Mixtures with 10 per cent of Castor Oil.

To illustrate a single phase of the usefulness of the test as described, Fig. 4 shows the stability values obtained from mixtures of a natural sheet asphalt sand and varying percentages of mineral filler with which was incorporated paving asphalt to the extent of 10 per cent by weight of the total mineral aggregate. This represents about 9.1 per cent of the total mixture. The sand was screened to pass a 10-mesh and be retained upon a 200-mesh sieve. It is seen that the addition of mineral filler in increasing amounts materially increases the stability of the mixture and that upon an equivalent weight basis the effect of hydrated lime, which is a very

finely divided product, is much more pronounced than that of the usual limestone dust filler.

This bears out the results of earlier investigations conducted by the authors in a study of the effect of various fillers upon the stability of mineral aggregates for sheet asphalt pavements irrespective of the asphalt cement. In these investigations, which have been published elsewhere,[1] 10 per cent of castor oil was mixed with the aggregate to form test specimens of similar size and shape to the asphalt briquettes described in this paper. The same type of test was applied to these castor oil mixtures but a test ring $1\frac{1}{2}$ in. in diameter was used instead of the $1\frac{3}{4}$-in. ring. The mixtures were made and tested in the same mold at room temperature and certain typical results obtained with a light traffic sand grading are given in Fig. 5 for the purpose of showing the similarity in trend of results obtained with the castor oil and asphalt mixtures. Of course the actual figures are not comparable as a smaller orifice was used in the castor oil tests.

It is believed that as soon as some definite relationship can be established between the two tests, the castor-oil method may be used to obtain quickly a vast amount of useful information relative to paving mixtures and that the test here described will confirm and add greatly to such information. Among the various factors affecting the stability of asphalt mixtures, upon which one or both tests are expected to shed considerable light, may be mentioned the following:

1. Effect of sand grading;
2. Effect of shape and texture of mineral particles;
3. Effect of various types and gradings of mineral fillers;
4. Effect of consistency and other characteristics of asphalt cement;
5. Effect of varying percentages of asphalt in various mixtures;
6. Effect of density of mix; and
7. Susceptibility of various mixtures to temperature variations.

As information such as this is obtained our specifications for materials and methods of constructing asphalt pavements may be gradually improved.

[1] Prévost Hubbard, "Research Work to Improve Asphalt Paving Mixtures," *Municipal and County Engineering*, December, 1924.

Prévost Hubbard and F. C. Field, "A Simple Method for Studying the Relative Value of Mineral Fillers for Asphalt Paving Mixtures," *Proceedings*, American Road Builders Association (1925).

A STABILITY TEST FOR BITUMINOUS PAVING MIXTURES

W. J. EMMONS[1] AND B. A. ANDERTON[2]

SYNOPSIS

The paper outlines in a general way investigations of the stability of bituminous paving mixtures which have been in progress at the Bureau of Public Roads for the past three years, and describes in detail a test that has been developed to correlate service behavior with laboratory determinations.

Specimens are compressed by means of a power tamping device and are 8 by 6 by 2¼ in. in size. The specimen, heated to 60° C. (140° F.) and confined in a mold, is subjected to pressure, a portion of the material being extruded from openings in the bottom and sides of the mold.

A series of curves are given which indicate that the test is sensitive to variations in the composition of bituminous mixtures and is adapted to the investigation of both the fine and coarse-graded types.

Three years ago the U. S. Bureau of Public Roads undertook the study of the stability of bituminous paving mixtures. The program adopted embraced both experiments upon especially constructed pavements and investigations in the laboratory upon compressed specimens, the objects being to develop a test which would render possible the determination of the relative stability of combinations of materials and ultimately to evolve a theory of proportioning.

Late in 1922, a circular track or roadway upon a smooth concrete fonudation was paved with 27 asphaltic-concrete mixtures of widely varying proportions. Traffic in the form of a loaded truck was applied, commencing in October. During the ensuing winter all of these mixtures showed no displacement, but with the advent of warm weather shoving began to occur. By the end of July, reference screws set in the mixtures showed that movements had occurred in all sections, virtually imperceptible in some and amounting to several feet in others. Although this test was designed primarily to determine relative resistances of mixtures under traffic for the purpose of correlation with the desired laboratory test, it was natural to attempt to draw direct conclusions from their behavior. Such conclusions, however, as might be indicated by the behavior of the track pavements, were confusing and evidently inconsistent.

During the winter of 1923–24, sawed specimens of these mixtures were subjected to compression tests and laboratory investigations were

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[2] Chemist, U. S. Bureau of Public Roads, Washington, D. C.

made upon the component sand mortars and coarse aggregates. Results of the latter in connection with the observed action of the track indicated the essential importance of developing the factors influencing the behavior of sand mixtures independently of the

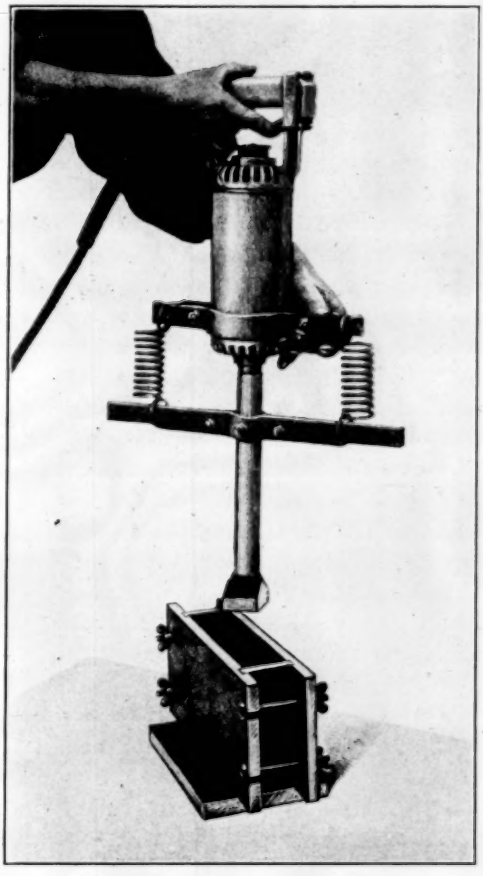

FIG. 1.—Mold and Tamping Apparatus.

presence of the coarse particles. Therefore, in July, 1924, the asphaltic concrete sections were removed and in their place was constructed a second series. This time 33 sections were laid, of which 5 were asphaltic concretes and 28 were sheet asphalts, the customary intermediate binder course under the latter being omitted. Following the installation of measuring devices and thermocouples, traffic was

started about the first of September and, after being suspended during the winter months, has now been resumed. Several of the mixtures have already displaced to some extent and it is anticipated that marked differences in their respective resistances to displacement will be shown by all of the sections before the warm months have passed.

Meanwhile, considerable has been accomplished in the way of developing a laboratory test by means of which it is hoped to reproduce in terms of laboratory results the demonstrated relative stabilities of the service test mixtures. A test has been developed which differentiates between mixtures of varying composition, and although it is as yet in somewhat tentative form it appears decidedly promising. Results would indicate that it is probably applicable to mixtures of both fine and coarse-graded types. The test involves the molding of a rectangular slab of the mixture and, through pressure exerted on one side of the confined specimen, extruding a certain portion through openings in the sides and bottom of the mold. Specimens used in this test are 8 in. long, 6 in. high and $2\frac{1}{4}$ in. thick. The mixtures are prepared by hand or mechanical mixing and are compressed in a collapsible steel mold, the sides of which are slightly greased. Compression is obtained by means of an electric hammer to which a square-ended tamper is fitted. The hammer being used works on the electromagnetic principle and strikes about 1500 blows per minute. As the heated composition is fed in small increments, the mold is gradually filled and finally struck off to a level surface by means of a hot iron. Five to eight minutes are required to mold a specimen by this method. Fig. 1 shows the apparatus used in the process of forming a specimen.

Although sheet asphalt mixtures can be molded into specimens under gradually applied pressures as imposed by a testing machine, it has been found that adequate compression cannot be obtained in the case of asphaltic concretes without fracturing the stone particles. With the trap rock aggregates used in the tests thus far, this method of tamping has resulted in very little breaking of the aggregate even when specimens of low voids have been made.

The strength or resistance to displacement as measured in the several methods of testing attempted is largely dependent upon the density to which the specimens are compressed. Consequently, at some time previous to testing the compacted specimen is removed from its mold and its specific gravity determined. From the known specific gravities and proportions of constituent materials the percentage of voids in a specimen is calculated.

It is apparent that the force which results in the longitudinal displacement of the bituminous surface is applied in the form of shear. The aggregate particles of the mixture when compressed under the roller occupy certain positions with relation to each other, and in combination with the bituminous binder possess to a greater or less degree strength to resist the forces which tend to change this structure. If, under conditions affecting the plasticity of the mixture, forces are applied of sufficient magnitude the aggregate particles move over one another and occupy new positions, giving rise to a different internal structure which may be less strong than the original. With greatly reduced strength, waving will ensue, or, under a condition of

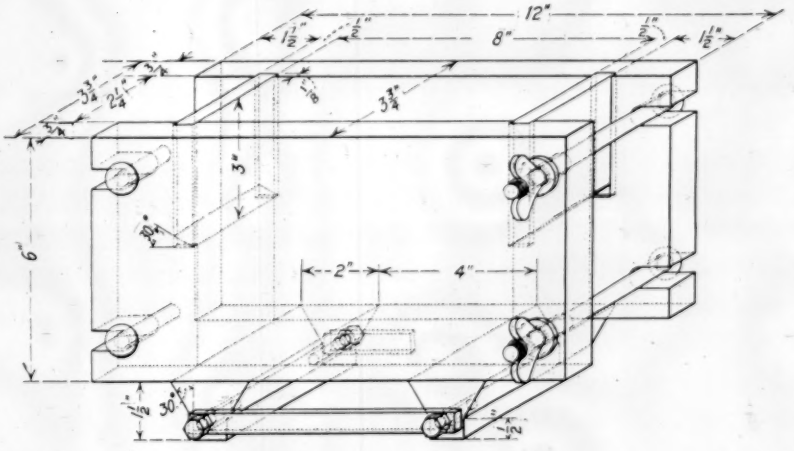

FIG. 2,—Diagram of Testing Mold and Base Plates.

extreme plasticity, rutting may result. Such a mixture must be regarded as unstable under the traffic, climatic and other local conditions to which it may have been subjected.

It is felt, therefore, that the proper test for stability should be essentially one of shear and that the force applied should in some manner cause an internal movement and re-arrangement of the particles without leaving them free to dissociate themselves completely during the progress of the test. The expedient of subjecting to pressure a specimen confined in a mold and forcing it through a single restricted orifice seems to fulfill these conditions and appears to be quite satisfactory for the testing of fine-graded mixtures. Considerable experimentation with such a single opening has already been accomplished by other investigators, using varied sand aggregates and fine mixtures. We feel, however, that this method is not likely

to prove successful for the use of mixtures containing coarse aggregate, since in specimens of a reasonable size the forced re-arrangement of the original position of the coarse fragments will result merely in the formation of a new and artificial internal structure with arching of the particles over the orifice, rendering a determination of the original resistance impossible. The test here described is intended to be applicable to the entire range of aggregate sizes, minimizing

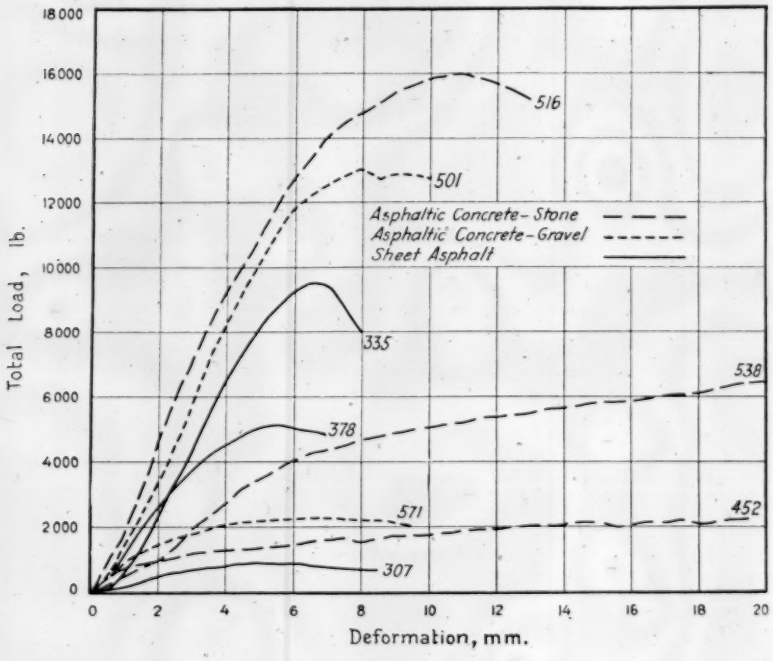

FIG. 3.—Typical Stress-Deformation Curves.

the objectionable arching action by permitting individual fragments considerable latitude in direction of movement.

The testing form or mold provides three openings for the extrusion of the mixture, the center 4 in. of the bottom and the lower 3 in. at each end. Fig. 2 illustrates the construction of the testing mold and its position upon bearing plates during the application of the load. The testing mold is identical with that used in forming the specimen except that full height end plates are necessarily used during that process.

After the specific gravity of the specimen has been determined,

the specimen is confined in the testing mold and immersed in a water bath maintained at 60° C. (140° F.). A generous time has thus far been allowed to raise the specimen to this temperature but it is probable that an immersion period of four hours is sufficient. The selection of 60° C. as the testing temperature was made as the result of thermocouple measurements of temperatures in the service test pavements. This temperature was reached during extremely hot periods when, of course, the pavement mixtures were also most susceptible to displacement.

When ready for testing, the mold containing the specimen is placed upon the properly spaced base plates in a metal tank containing water at 60° C., which is in turn placed upon the weighing table

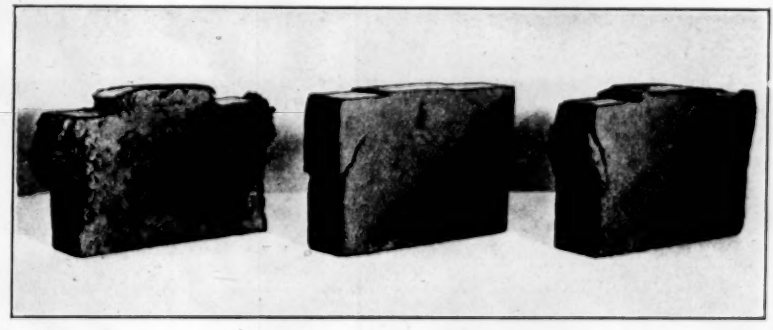

FIG. 4.—Specimens After Testing.

of a 20,000-lb. compression machine. The load is applied through a spherical bearing block which rests upon a steel plate having a $\frac{1}{16}$-in. clearance around the sides of the mold. The machine is run at its slowest testing speed, which with this machine lowers the head at the rate of 0.073 in. per minute. The loads are observed at 0.5-mm. intervals of deformation.

Fig. 3 shows typical stress-deformation curves resulting from tests of both sheet asphalt and asphaltic-concrete mixtures. In all tests of the former type a very definite maximum load is reached, after which deformation of the specimen progresses under a decreasing load. This statement is also true of the asphaltic-concrete mixtures tested thus far, provided that in the process of molding a specimen a good density is obtained. Specimens of this type which are not well compressed frequently do not exhibit definite points of failure, although indications evidenced by periods of continued deformation under constant load are often noted. This situation is, however,

not likely to prove disturbing since it is improbable that much useful information can be obtained from specimens which are not compressed to densities comparable with those attained in construction work. Poorly compressed specimens were, however, purposely made to determine the limits of usefulness of the test being developed. In Fig. 3, specimens Nos. 452 and 538 were poorly compressed, whereas specimen No. 516 was molded to a high density. The differences in

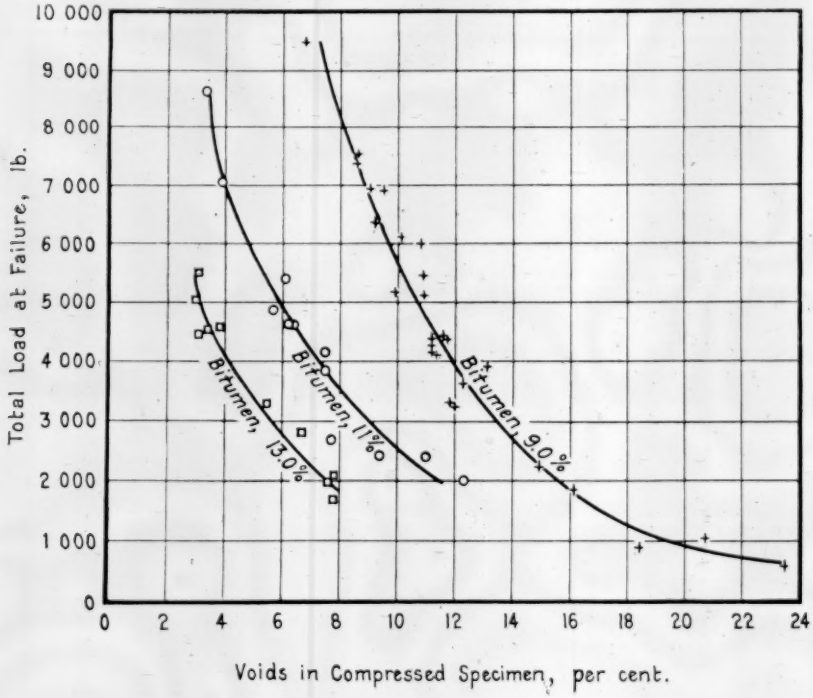

FIG. 5.—Relation Between Voids and Strength with Changes in Bitumen; Sheet Asphalt.

behavior due to this condition are clearly shown by the shapes of the curves.

Fig. 4 illustrates the condition of tested specimens after they are removed from the testing molds. They are shown here in an inverted position. The center specimen is composed of a sheet asphalt mixture while the end ones are of asphaltic concrete. In the vicinity of the mold orifices the internal structure of the tested specimens is entirely disrupted, and when the test is carried as far as was done in the case of the specimen shown at the left, the extruded material crumbles readily. No fracture of coarse aggregate has been noted in extruded material resulting from deformation under test.

Figs. 5, 6, 7 and 8 show the relation between voids in a large number of the specimens and their respective maximum strengths, or in the case of the lower density asphaltic-concrete specimens, the load at which the form of the stress-deformation curve seems to indicate a reduced stability. The proportions used in these mixtures were varied in such a manner as to indicate the sensitivity of the test. A Potomac River sand possessing a grading commonly regarded as

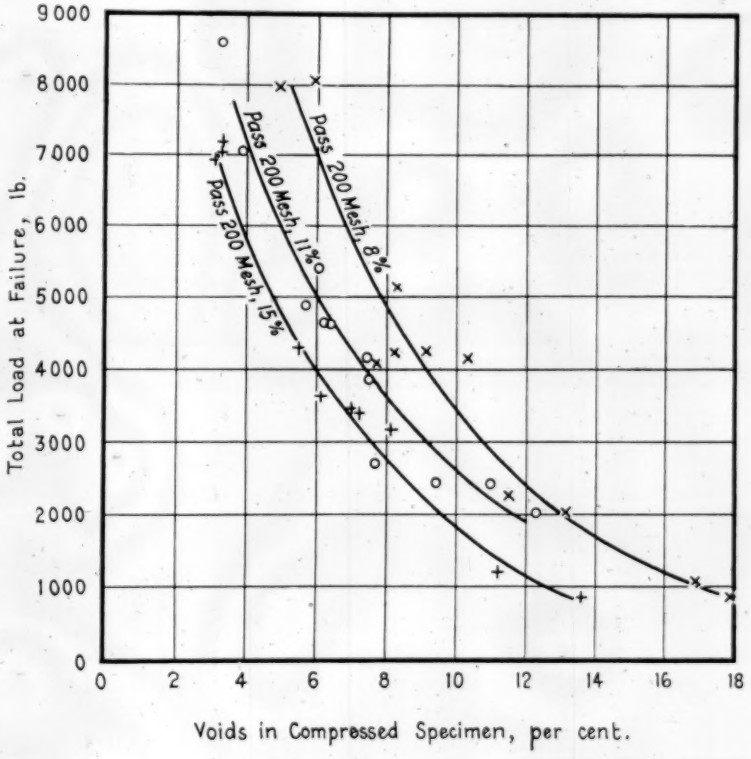

FIG. 6.—Relation Between Voids and Strength with Changes in Filler; Sheet Asphalt.

suitable for light-traffic sheet asphalt construction was used in all of these specimens. Limestone dust served as a filler and a Mexican petroleum asphaltic cement of 55 penetration was used throughout the work. Trap rock graded from $1\frac{1}{2}$ to $\frac{1}{4}$ in., but containing a considerable number of fragments whose greatest dimension was 2 in., was used in the stone asphaltic concretes. Gravel graded between the same limits constituted the coarse aggregate in the series whose test results appear in Fig. 8.

Figs. 5 and 6 indicate differences in the strength of the sheet asphalt mixtures resulting from changes in the bitumen and filler contents, respectively. Fig. 7 shows the effect of variations in bitumen content upon a certain asphaltic concrete. In these latter mixtures the bitumen contents were computed by assuming that the stone required 1.25 per cent of bitumen and that the sand and dust carried

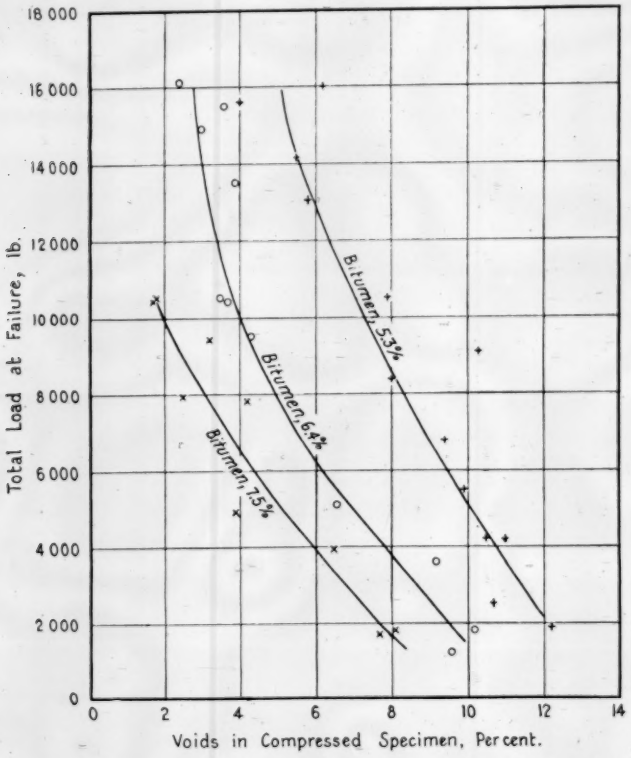

FIG. 7.—Relation Between Voids and Strength with Changes in Bitumen; Stone Asphaltic Concrete.

9, 11 and 13 per cent of bitumen in the three series indicated as containing 5.3, 6.4 and 7.5 per cent of bitumen, respectively, thus conforming to the percentages used in the three sheet asphalts shown in the curves of Fig. 5. The total aggregate mixture of the asphaltic-concrete specimens is composed of equal parts retained upon and passing a 10-mesh sieve. The single series of gravel asphaltic-concrete specimens are of the same proportions as the 6.4-per-cent bitumen stone asphaltic concretes of Fig. 7.

These results should not be taken as definite measures of the stability of mixtures so proportioned. Data thus far obtained are much too meager to be so used and the correlation between the behavior of the service test sections referred to at the beginning of this paper and the results of the strength tests remains to be accomplished before definite interpretation of the results can be given and the true value of the test confidently asserted.

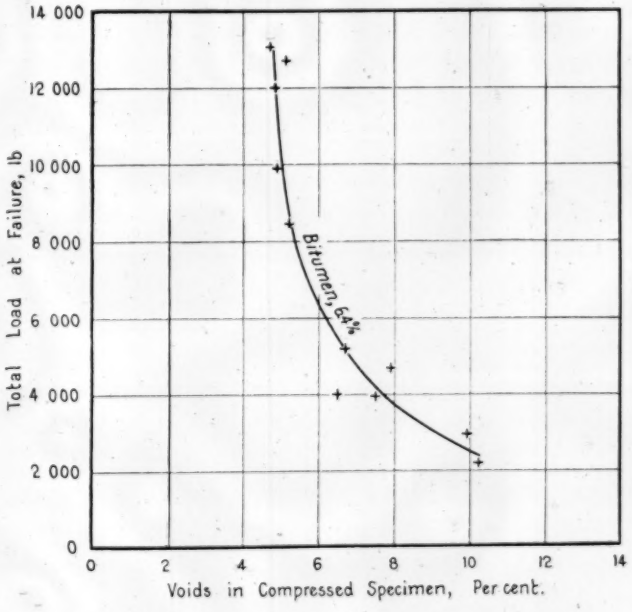

FIG. 8.—Relation Between Voids and Strength; Gravel Asphaltic Concrete.

In conclusion, a summation of the work completed at present indicates the following:

1. The compaction of mixtures by power tamping is a convenient and efficient method of forming specimens of bituminous paving mixtures, particularly those containing coarse aggregate.

2. An extrusion test of this character is sensitive to variation in composition of bituminous paving mixtures and is well adapted to the study of the resistance to displacement of both fine and coarse-graded mixtures.

A DEFORMATION TEST FOR ASPHALTIC MIXTURES

By H. M. Milburn[1]

Synopsis

This paper describes a deformation test intended for obtaining data to assist in determining the proper proportions to use of a given asphalt and a given mineral aggregate in order that the resulting mixture may best withstand high temperature. A machine is described for subjecting specimens of the mixture to a constant load at a constant temperature for a definite length of time. The decrease in height of the specimen is determined. Some of the results obtained with the machine are given.

The machine and tests described in this paper are intended for the purpose of ascertaining the effect of varying the percentage of asphalt in asphaltic mixtures by subjecting compressed specimens of the asphaltic mixtures to a constant load at a constant temperature for a definite time and ascertaining the decrease in height of the specimens. The machine for obtaining the decrease in height of the specimens is designated in this paper as a deformation machine and the decrease in height of the specimens is designated as the deformation.

Deformation tests at high temperatures will, it is believed, be of value in assisting to determine the proper proportions to use of a given asphalt and a given mineral aggregate in order that the resulting mixture may best withstand high temperature.

Description of the Machine

The deformation machine, shown in Fig. 1, consists of the frame of a Vicat machine supporting a cylindrical rod in a vertical position. A cup for holding shot is attached to the upper end of the rod, the lower end being fitted into a cylindrical shoe 1 in. in height by $1\frac{1}{4}$ in. in diameter. Sufficient shot is placed in the cup so that the weight of the shot, cup, rod and shoe is 5 kg. The decrease in height of the specimen is obtained by the use of an Ames dial reading to 0.001 in. attached to the frame in such a manner as to record the decrease in height of specimens, the foot of the Ames dial resting on a metal strip fastened to the rod of the machine. The size of the specimens tested is 1 in. in height by $1\frac{1}{4}$ in. in diameter. The machine is about 8 in. wide and 22 in. high over all. The deformation results given in this paper were obtained with this machine. Recently the machine has been redesigned in order to make it more compact, but in principle

[1] Chemist, U. S. Bureau of Public Roads, Washington, D. C.

it is the same as the machine just described. This new machine, shown in Fig. 2, differs primarily from the former machine in that two vertical parallel rods act as bearings and the cup containing the shot is replaced by a rectangular weight. The machine is about $7\frac{1}{2}$ in. wide and 12 in. high over all.

PREPARATION OF SPECIMENS

In the preparation of the specimens, four different asphalts were used. Mixtures of each asphalt and a mineral aggregate consisting of 88 parts by weight of an asphalt sand and 12 parts by weight of a mineral filler were made, containing from 8 to 15 per cent of bitumen by weight in increments of one per cent in three series and from 2 to 15 per cent of bitumen by weight in increments of one per cent in one series. The mesh compositions of the mineral aggregates used in the three series containing specimens of 8 to 15 per cent of bitumen and the mesh composition of that used in the series containing specimens of 2 to 15 per cent of bitumen are given in Table I.

TABLE I.—MESH COMPOSITION OF MINERAL AGGREGATE.

				Three Series Containing Specimens of 8 to 15 per cent Bitumen	Series Containing Specimens of 2 to 15 per cent Bitumen
Passing	$\frac{1}{4}$-in.	Retained on	10-mesh...	0.7 per cent	0.3 per cent
"	10-mesh	"	20-mesh...	6.0 "	6.9 "
"	20-mesh	"	30-mesh...	8.7 "	9.8 "
"	30-mesh	"	40-mesh...	11.9 "	10.3 "
"	40-mesh	"	50-mesh...	10.2 "	17.2 "
"	50-mesh	"	80-mesh...	25.1 "	23.4 "
"	80-mesh	"	100-mesh...	7.5 "	7.0 "
"	100-mesh	"	200-mesh...	12.3 "	10.0 "
"	200-mesh			17.5 "	15.1 "

In the preparation of the mixtures, the asphalts and mineral aggregates were heated separately to a temperature of 300 to 325° F. and then thoroughly mixed in the proportions desired by hand, using a trowel or large spatula. In the making of the specimens a hollow steel mold $2\frac{1}{2}$ in. in diameter and 3 in. in height was used. The diameter of the hole in the mold is $1\frac{1}{4}$ in. The mold which had been previously heated was placed on a metal plate resting on the base of an Olsen testing machine and sufficient of the mixture to form a specimen substantially 1 in. in height was added. A cylindrical steel plunger $3\frac{1}{2}$ in. long and $1\frac{15}{64}$ in. in diameter was then placed in the mold. Pressure was then applied until the beam balanced at

2000 lb., using high speed, then the balance weight was set at 10,000 lb. and the pressure applied at low speed until the beam just balanced. The pressure was then released and the specimen removed from the mold. The removal of the specimen was effected by placing the mold

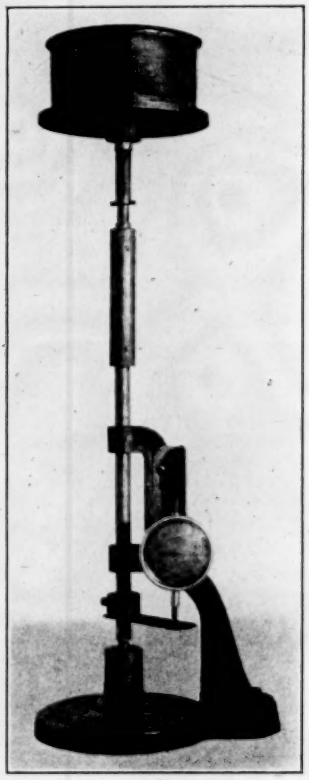

FIG. 1.—View of Deformation Machine.

on a metal tube of sufficient height and diameter to allow free passage from the mold and forcing the specimen out by applying pressure to the plunger. The specific gravities of the specimens were determined by the displacement method.

While in these four series a pressure of 10,000 lb. was used exclusively, the pressure that was originally tried was 20,000 lb. as this pressure had previously been used with another mineral aggregate and specimens whose specific gravities seemed reasonable were obtained.[1] In these series, however, specific gravities which appeared

[1] The results of this investigation were embodied in a Progress Report of a Sub-Committee of the Committee on Tests, American Association of State Highway Officials for 1921.

abnormally high were obtained with low-bitumen content specimens. Furthermore, the specific gravities in general decreased from a maximum for the 8-per-cent bitumen specimens to a minimum for the ones containing 15 per cent of bitumen. Analyses of the 8-per-cent bitumen specimens showed the mineral aggregate to contain 27.9 per cent of material passing the 200-mesh sieve, whereas the original mineral aggregate contained only 17.5 per cent. This increase in

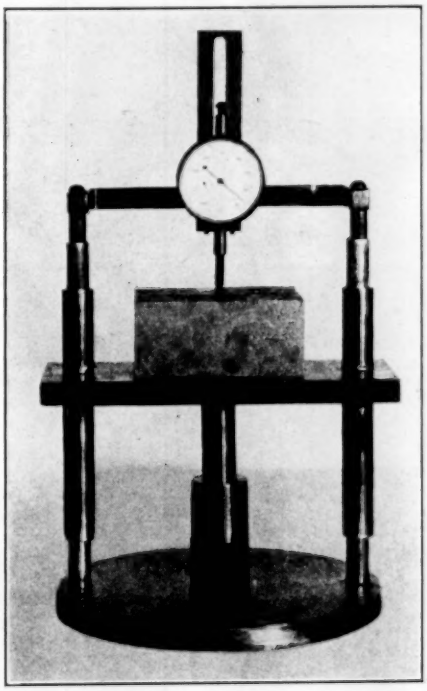

FIG. 2.—View of New Deformation Machine.

very fine material indicates that at this pressure crushing of the mineral aggregate had occurred. It has been a matter of record for many years[1] that the voids in a sand decrease continuously to a certain point with the increase of mineral filler. It can be assumed, therefore, that the increase in fine material in the specimen had decreased the voids, resulting in high specific gravities.

TESTING SPECIMENS FOR DEFORMATION

The deformation machine was placed in an oven and heat so applied that a temperature of 60° C. was registered by a thermometer

[1] "The Modern Asphalt Pavement," by Clifford Richardson, 2nd edition, p. 82.

embedded in a specimen on the base of the machine. The specimen to be tested was kept in the oven for one hour. The load was then applied and the deformation of the specimen was obtained by noting the original reading of the Ames dial and the reading at the end of

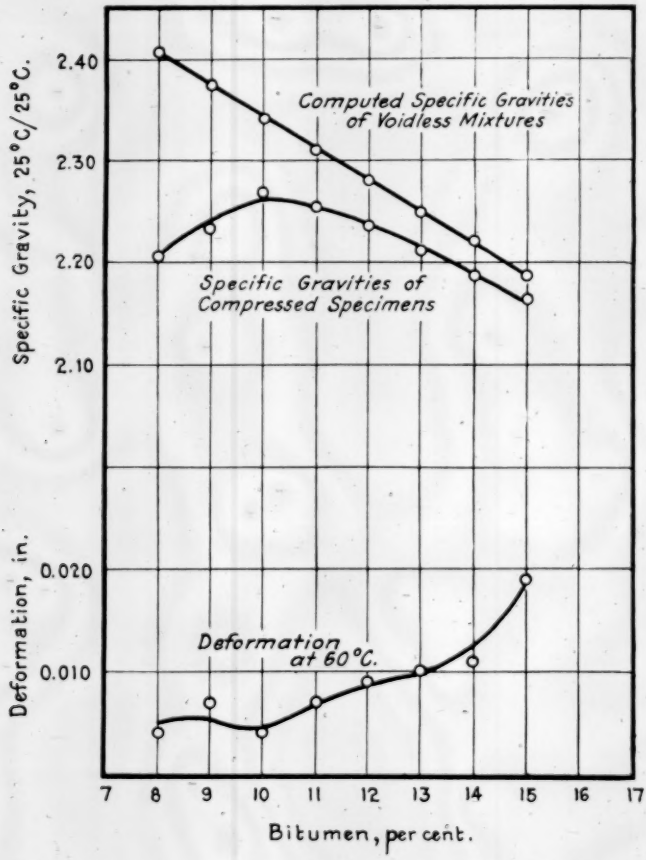

FIG. 3.—Specific Gravity and Deformation Results on Mixtures in Which Asphalt L Was Used. Penetration, 25° C., 100 g., 5 seconds, 53.

2 hours. The specific gravity and deformation results obtained on the specimens of the four different series of mixtures are given in Figs. 3, 4, 5 and 6. In the deformation tests, the results of specimens which collapsed during the test are designated as total and no attempt was made to measure the deformation of such specimens.

DISCUSSION OF RESULTS

In Figs. 3, 4, 5 and 6 the asphalts used have been designated respectively as Asphalt L, J, I and K. In Fig. 3 the deformation results decrease from the 15-per-cent bitumen specimen to and including the one containing 10 per cent of bitumen, then an increase

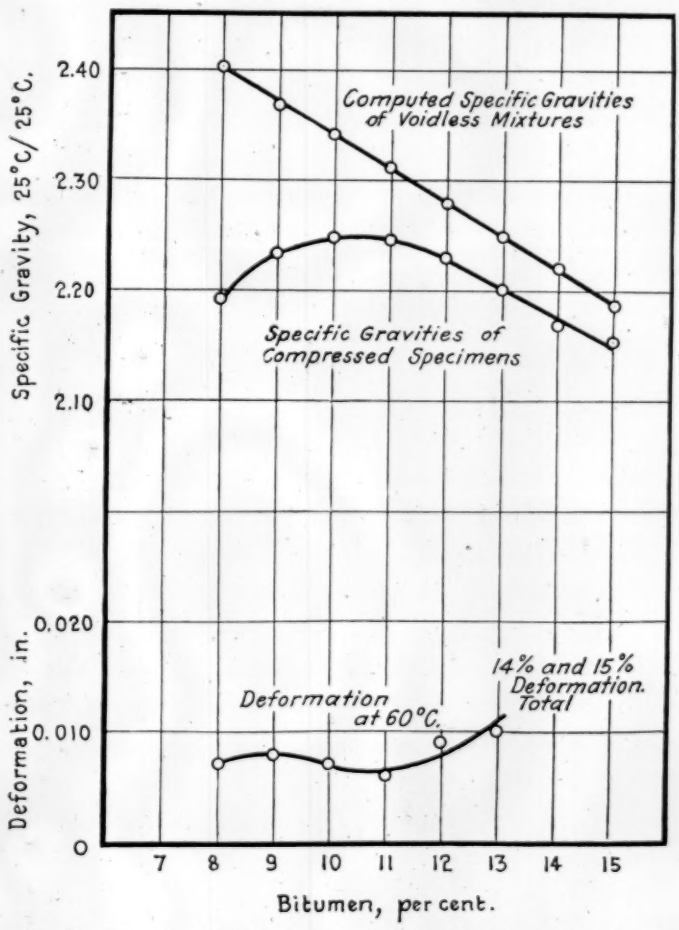

FIG. 4.—Specific Gravity and Deformation Results on Mixtures in Which Asphalt J Was Used. Penetration, 25° C., 100 g., 5 seconds, 53.

occurs for the 9-per-cent bitumen specimen with a subsequent decrease for the 8-per-cent specimen. Fig. 4 shows that the deformation decreases from the 15 and 14-per-cent bitumen specimens to the 11-per-cent one, inclusive, and then increases up to and including the

9-per-cent specimen, with a subsequent decrease for the 8-per-cent specimen. Fig. 5 shows that the deformation decreases from the 15-per-cent bitumen specimen to and including the 10-per-cent specimen. An increase in deformation then occurs up to and including the 8-per-cent bitumen specimen, a decrease then occuring with subsequent increase resulting in total deformation with the 2-per-cent bitumen specimen. These results appear to indicate that there are

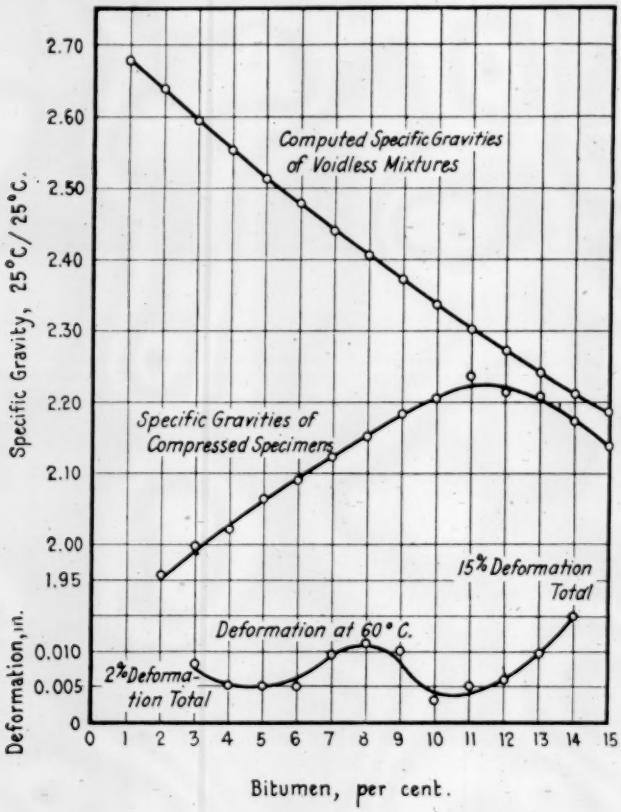

FIG. 5.—Specific Gravity and Deformation Results on Mixtures in Which Asphalt I Was Used. Penetration, 25° C., 100 g., 5 seconds, 36.

two fields of comparatively low deformations, one field embracing specimens of high specific gravities and the other field embracing specimens of very low specific gravities. In a correlation of specific gravity determinations with deformation determinations for the purpose of ascertaining the proper amount of bitumen to use with a given mineral aggregate, the field of deformation results embraced in the field of high specific gravities should, I believe, be the results to be given greatest consideration.

The deformation results in Fig. 6 show a decrease from a maximum at the 15-per-cent bitumen specimen down to and including the lowest bitumen content specimen, namely, 8 per cent. Consequently, to obtain data relative to the best percentage of bitumen to

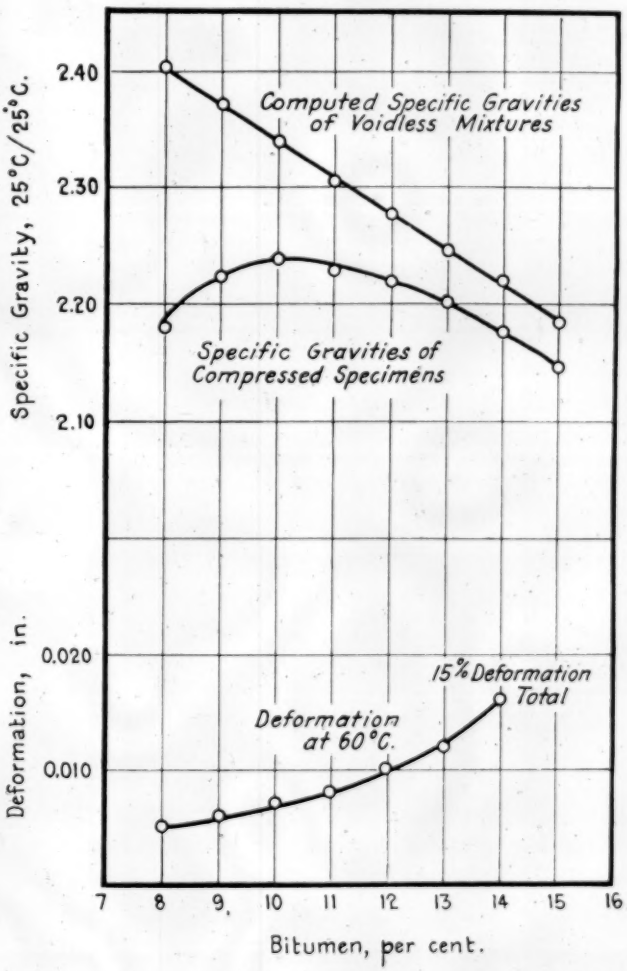

FIG. 6.—Specific Gravity and Deformation Results on Mixtures in Which Asphalt K Was Used. Penetration, 25° C., 100 g., 5 seconds, 51.

use it would be necessary to correlate the results of the deformation tests with other physical tests and I believe that specific gravity results should be given considerable weight in a correlation of results when attempting to judge of the proper proportioning of a mixture.

The specific gravity results of the specimens given in Figs. 3, 4, 5 and 6 all show the same general characteristics, namely, they all increase in general up to a maximum and then decrease. As no voidless specimens were obtained, the specific gravity results, of course, in no instance are as great as the calculated specific gravities of the voidless mixtures. It is of interest to note that in no series does the maximum specific gravity result represent a specimen containing the minimum percentage of voids, the specimen of lowest voidage in each series being a specimen with lower specific gravity than the maximum.

CONCLUSIONS

Results seem to indicate:

1. That the effect at high temperatures of varying the proportions of an asphalt and a mineral aggregate of the type used in the wearing course of a sheet asphalt pavement can be ascertained by subjecting compressed specimens of the mixture to a constant static load at a constant temperature for a definite time.

2. That by a correlation of the specific gravity results of the compressed specimens with the deformation results, data are obtained relative to the proper proportions to use of a given asphalt and a given mineral aggregate in the design of asphalt mixtures of the type used in the wearing course of a sheet asphalt pavement. It is realized, however, that a deformation test at a high temperature may not necessarily indicate the proper proportioning of the mixture in order that it may best withstand low temperatures. Under such conditions it would seem necessary to develop physical tests for mixtures at a low temperature for correlation with specific gravity and deformation results.

DISCUSSION ON STABILITY OF BITUMINOUS PAVING MIXTURES

MR. M. H. ULMAN[1] (*presented in written form*).—The Pennsylvania Department of Highways collaborated in a series of experiments with a view of developing apparatus which might prove of value in arriving at a laboratory means of determining the stability of fine-graded bituminous mixes in which series one of the machines used was similar to that described by Mr. Milburn. In our investigation, five different asphalts of practically the same consistency were used and two sands of varying grain texture and conforming to recognized grading for light traffic and heavy traffic pavements. The sands, however, had all grains smaller than 200 mesh removed prior to mixing. **Mr. Ulman.**

A brief description of our procedure is as follows:

A. TESTS ON MATERIALS

1. *Asphalt Cement.*—The asphalt cements were tested in accordance with A.S.T.M. requirements for conducting the following tests: Penetration at 77° F.; penetration at 32° F.; penetration at 115° F.; specific gravity at 77° F.; evaporation loss, 5 hours, at 325° F.; carbon residue penetration at 77° F.; ductility at 77° F.; solubility in carbon disulfide; solubility in carbon tetrachloride; toughness.

In addition, toughness determinations were made at 39.2° F. by means of the Page impact machine on cylinders measuring 1 by 1 in.

2. *Filler.*—Mechanical analysis and specific gravity determinations were made.

3. *Sand.*—Mechanical analysis, specific gravity and microscopic examination.

B. MIXING

All individual mixes were made in amounts of 1000 g., and prior to molding analyses were made in order to determine if the actual mix conformed to the theoretical or desired proportioning. The variables were five different asphalts, two different sands, the one a straight run New Jersey bank sand, the other a blended sand, consisting of lake, river and bank. The first series consisted of constant filler content at 14 per cent with the asphalt varying from 8 per cent to 15 per cent in increments of 1 per cent. The other series consisted of constant bitumen at 11 per cent and a varying filler content of from 8 per cent to 15 per cent in increments of 1 per cent.

C. MOLDING

Fourteen cylinders measuring 1¼ in. in diameter and 1 in. in height were molded from each mix, under a pressure of 15,000 lb. in electrically-heated molds. This required a total of 2240 cylinders.

[1] Assistant Engineer of Materials, Pennsylvania State Highway Department.

D. TESTING

Mr. Ulman. The following tests were conducted:

Specific Gravity.—The specific gravity was determined on all cylinders both by the displacement method and by calculation on the basis of weight and volume. The height was determined by means of a micrometer.

Voids.—The voids were calculated.

Toughness at 39.2° F. and at 77° F. (3 specimens at each temperature).—The Page impact machine was used. The blows were imparted by a 2-kg. weight falling from heights increased in increments of 1 cm. until fracture occurred. Results calculated in terms of kg-m.

Deformation, Impact at 140° F. (2 specimens).—The specimens were maintained at 140° F. and tested in the Page impact machine in which a blunt headed plunger was substituted for the spherical one. The method was to permit 20 blows of 2 kg. to fall through a distance of 20 cm. Examinations of the specimens were made after every five blows to determine if fracture had occurred. Deformation was measured after the twentieth blow or when failure occurred if the specimen failed prior to the completion of the test.

Deformation, Static at 140° F. (2 specimens).—Modification of Milburn Method. Three kilogram load acting directly on the specimen for two hours. Deformation was recorded every first and second hour. In case of failure the deformation at the time of failure was recorded together with the time in which failure occurred.

Absorption (2 specimens).—Absorption determinations made weekly during the first month, then at the end of second, fourth, eighth and twelfth month. Two specimens were held in reserve for check purposes.

We have developed 110 individual graphs on the basis of these tests in order to illustrate the results in combinations pertinent to the desired information.

In interpreting the results of this investigation we find:

Toughness at 39.2° F.—The toughness of the asphaltic cement at 39.2° F. bears a relationship to the toughness of the mixture at this temperature. In other words, if the toughness of the asphalt is high the test on the mix will also give proportionately higher values than in the mixes in which the asphalt cement used had a low toughness.

Impact at 77° F. is the maximum temperature at which this test may be conducted, for if the temperature is in excess of that specified, deformation instead of fracture of the specimen will occur.

Voids can possibly run as high as 10 per cent and not affect the stability of the mixture as determined in these tests, for in the course of this investigation mixes having void contents as high as 7 per cent proved of equal and in some instances greater stability than those of relatively lower void content.

Absorption.—Results obtained showed little variation except in the case of the lowest bitumen content and its main value is in determining if mixes had been burnt in heating.

Filler.—All tests indicate that a variation in filler between 8 per cent and 15 per cent, with constant bitumen content at 11 per cent, has very little effect on stabilizing the mixtures. The susceptibility of the asphalt cement to temperature change has a greater effect than the mechanical analysis of the mineral aggregate. Mr. Ulman.

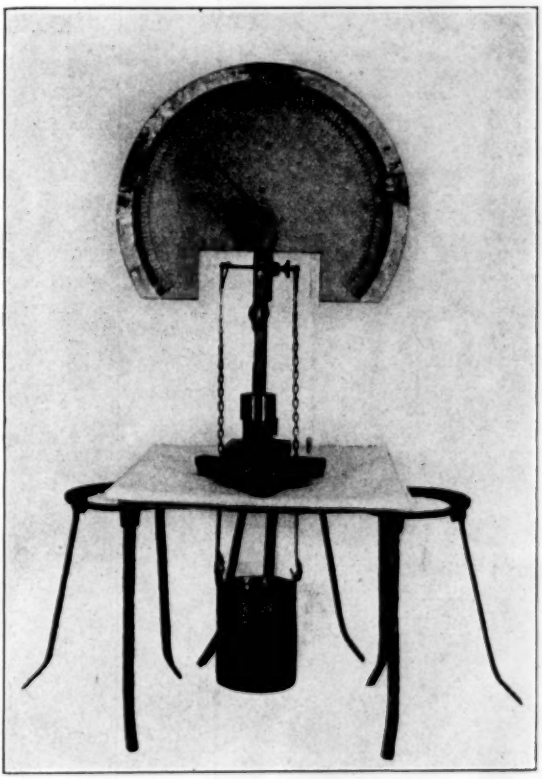

FIG. 1.—Apparatus for Static Deformation Test.

No single test, as enumerated, advances complete information as to the stability of the mixture, for it requires a study of the data obtained in the various tests. This is best illustrated where blown asphalt was used, for the results obtained gave the best values except in the impact deformation when complete failure occurred under ten blows, or when half the test had been completed, while with the other asphalts it was possible to measure deformation after applying twenty blows, although varying amounts of deformation was obtained in these cases.

Mr. Ulman. The machine used for the formation test is shown in Fig. 1 and is an adaptation of Mr. Milburn's original design. We, however, used a total load of 3 kg. instead of 5 kg. In Mr. Milburn's mixtures he varies both the asphalt and filler in his method of making additions of asphalt cement, while in our work the filler is maintained at a constant specified amount and the varying amounts of asphalt which were added was compensated for by varying the sand content, which makes simpler the calculations covering the mechanical analysis of the mineral aggregate. Mr. Milburn states that his mold was heated prior to molding the specimens. We have found that more uniform results are obtained by the use of an electrically heated mold in which

FIG. 2.—Mold for Compressing Specimens.

the desired temperatures are uniformly maintained by a resistance bank. The type of molds used is illustrated in Figs. 2 and 3.

We have found that more accurate data are obtainable by expressing the results in the time required for the plunger or shoe to penetrate the specimen.

The molding load of 15,000 lb. used was an intermediate load between those of Mr. Milburn's two series of investigations. Based on analyses made after molding the specimens, we found that fracture or change of mechanical analysis is a function of grain strength in mixtures ranging from 8 to 10 per cent of asphalt, for in those mixtures containing greater amounts of asphalt the binder probably acts as a cushion and prevents fracture. One of the sands used consisted of quartz and sandstone, while the other sand was entirely quartz and no fracture of grains was obtained in the latter mixes. There was no appreciable difference in tests, due to the slight change in mechanical

analysis, but it would be preferable to use the 10,000-lb. pressure as a general procedure in consideration of variations in grain strengths of sands which may be encountered. **Mr. Ulman**

We also found that the specific gravity of the cylinders were highest with lowest asphaltic cement content and decreased with increase of asphalt cement. In Mr. Milburn's curves, he shows that

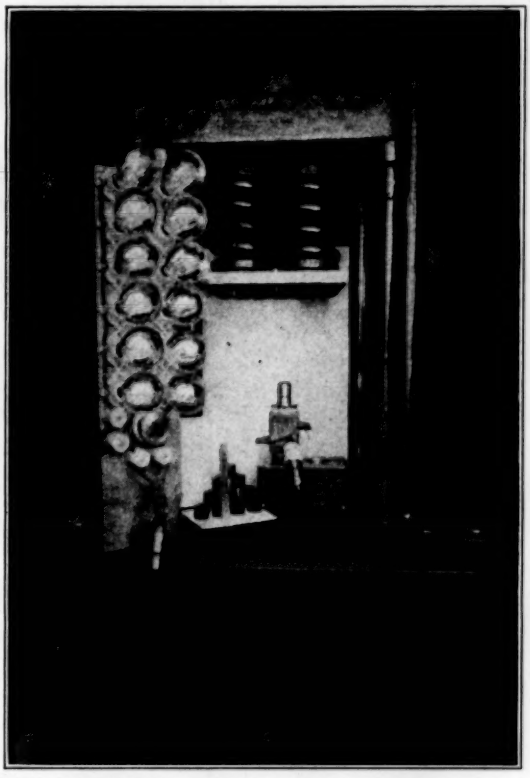

FIG. 3.—Mold for Compressing Specimens.

his highest specific gravities were obtained with a bitumen content of between 10 and 11 per cent, while in our results the peak specific gravity was always obtained with the 8 per cent bitumen content. Further, we did not find that there were two fields of low deformation corresponding to high and low specific gravity, for our specific gravity decreased with increase of asphalt and we found that the deformation practically followed a straight line and increased deformation occurred between 11 and 14 per cent of asphalt, depending on the susceptibility

Mr. Ulman. of the asphalt cement used. In consideration of these results it would appear that further investigation would be of value in order to determine if maximum stability would be obtained using a low percentage of asphalt but of softer consistency than what is now recognized for standard practice.

This method of test appears to be of value in obtaining load point or the maximum amount of asphalt which may be used with safety but does not, in itself, furnish full information for it is essential to correlate results, as Mr. Milburn suggests, with other tests in which may be mentioned the impact deformation which appears to be one of the tests essential for correlation.

Mr. Smith. MR. F. P. SMITH.[1]—These papers represent a very distinct and a very valuable contribution toward methods of studying in the laboratory the effects of varying the composition and character of different bituminous mixtures. As Mr. Hubbard rightly pointed out, it is, however, dangerous at this time to attempt to draw any definite conclusions regulating practice based upon the tests devised and the results which have so far been obtained. This perhaps can very clearly be illustrated by a theoretical consideration of two possible methods of minimizing permanent deformation under traffic.

From a theoretical standpoint, without any very elaborate investigation, it is easy to reach the conclusion that deformation under traffic stresses can be eliminated by making the mixture more rigid. As we make the mixture more rigid, however, we run into other very serious difficulties. The greater the rigidity, the more we depart from what has heretofore been the general and valuable characteristic of a bituminous surface mixture, namely, its pliability, its plasticity, its capacity to absorb and resist shock. The pavement also becomes more liable to fracture under impact, and at low temperatures it will grind away under the attrition produced by heavy traffic.

Thus we see that it is possible to resist deformation under heavy traffic if that is the only end to be considered, by producing a mixture with such a hard asphalt or bituminous binder that the mixture becomes a rigid one at normal temperatures and more nearly approaches in characteristics a portland-cement concrete.

On the other hand, if we have a bituminous surface of such fluidity or plasticity that any deformation produced by the impact of traffic will heal itself by reason of the fluidity of the mixture, we also arrive at a point where we are justified in stating that it is possible to avoid permanent deformation of our pavement surface by an extremely fluid and plastic surfacing medium. Such a conclusion is,

[1] Member of Firm, Dow and Smith, Paving and Chemical Engineers, New York City.

however, unsound from a practical point of view, because such a mixture would be so soft that the tractive resistance would be too great and the pavement would not be stable on grades. Mr. Smith.

It is apparent, therefore, that there are a great number of practical considerations connected with the manufacture and the laying of bituminous pavements which must be very carefully borne in mind before it is safe to draw any definite conclusions from any specific test or series of tests made in the laboratory. Such tests, to be of any real value, must in every instance be carefully checked up and correlated by service results under normal traffic conditions.

While I desire at this time to voice my appreciation of the work that has been done and that has been set forth in the papers to which we have listened, I would also like to sound a warning against attempting to draw any too definite or too wide-spread conclusions based on laboratory tests alone.

MR. H. L. HOWE, JR.[1]—About three years ago, we made an investigation to show the relative merits of different types of asphalt pavement filler. This necessitated the development of a test for stability and we used, with considerable success, a ball penetration test similar to the Brinell hardness test in steel work. Mr. Howe.

We hope to apply this ball test this summer directly to the pavement in the field by means of a small hydraulic jack equipped with a gage to measure the pressure applied on a $1\frac{3}{4}$-in. diameter ball. This should be successful depending, of course, upon whether or not we can tie up the stability of the pavement with the variation in temperatures of the pavement which would also have to be taken in such a field test.

MR. G. H. PERKINS.[2]—I want to encore Mr. Smith's statement or rather warning to not depend too much on laboratory tests. It has been our experience with a good many different mixtures that laboratory tests are an indication of what to expect on the road but nothing more, except that laboratory tests are, of course, excellent in control of manufacture. But these tests for stability, I believe, are not measuring the effect of the kind of strains we get on the road. Mr. Perkins.

For instance all of these tests measure the effect of a compressive load increasing at a definite rate, from zero up to the load required to cause a shear or flow within the mixture. Such a method of loading may be applicable for determining the type of mixture which will best withstand the effect of standing vehicles, but the traveling public

[1] Mechanical and Electrical Engineer in Charge Municipal Testing Laboratory, Department of Engineering, City of Rochester.

[2] Vice-President, Warren Brothers Co., Boston.

Mr. Perkins. is more concerned with deformations in the traveled way, and to measure the effect of moving vehicles I think we need a test which is somewhere between impact and static load.

While it is true that when a motor truck running along a bituminous pavement which had started to wave, does jump, the wheels leaving the pavement and then striking it, giving pure impact, the wheels of the average automobile do not. Again while the impact of a heavy motor truck unquestionably damages the foundation and thereby produces deformations in the pavement as a whole by depressing surface and foundation together, I question whether it often causes deformation within the surface mixture itself.

The average vehicle is in contact with the surface continuously and in passing over any one spot imposes on that spot, not impact, but a strain which rises from zero to a maximum and decreases again to zero almost instantaneously. The pressure of the wheel tends to push the mixture forwards and sideways but not backwards.

For instance, many bituminous roads under medium weight traffic—that is, with not very many motor trucks, but mostly pleasure automobiles—will remain perfectly smooth for two or three years and then suddenly in a hot spell start to wave or produce a washboard. In other words, the deformation is the accumulated effect of an enormous number of small instantaneous loads or pushes.

I think you will find, also, that in any wide street without car tracks the deformations are on the quarter, largely due to the side-thrust of traffic. The pavement is being pushed forward and at the same time is being pushed over toward the gutter. These deformations do not always appear directly under the spot which is struck by the traffic. For instance, I remember seeing the effect of the first set of traffic tests at Arlington, described by Mr. Anderton, where in many cases the bituminous-concrete mixture had been pushed forward several feet, yet a straight edge laid in the line of traffic showed no vertical deformation at all. On the other hand, the pavement about a foot to one side of the wheel tracks showed a ridge several inches high, but practically no lateral movement.

We are now starting some tests with a machine that we hope will be able to reproduce a series of pressures on the pavement surface without giving either impact of an actual blow or static pressure for any length of time.

I should like to ask the gentlemen who have been describing these tests whether or not they have sawed off the ends of any of their specimens and determined the specific gravity of the end pieces as compared with the center of the specimen. We have found it

almost impossible, in making small blocks of pavement, with any method of tamping, to obtain the same compression around the edge of the mold as we get in the center. **Mr. Perkins.**

MR. B. A. ANDERTON.—Answering the question of Mr. Perkins, I can say that we have cut up into nine pieces a few of the small specimens I have mentioned which have dimensions of 3 by 4 by $1\frac{1}{8}$ in. Three of these specimens were cut and the variation in the voidage between the nine sections ranged somewhere from $1\frac{1}{2}$ to 4 per cent, depending upon the average percentage of voids in the specimens. There was no regular variation in the voidage depending upon the position of the particular section but practically the lowest voids were obtained in the center of the specimen. We have not cut any of the larger specimens, which probably would show more variation than the smaller specimens. **Mr. Anderton.**

MR. ULMAN.—In reply to Mr. Perkins' question, we have not made many specimens with the asphaltic-concrete mixture, but we used a type of mold similar to the one I described, in this discussion, which is electrically heated. It is wrapped with nichrome wire, so that a uniform temperature is maintained at all times. The fit of the plunger is compensated for by the expansion in the cylinder upon heating, so a very tight fit just overcoming the frictional resistance is obtained. With the load we used and on the basis of the numerous (I should say 2200) specific gravity determinations made on all cylinders that were molded, the variations were practically negligible on each fine-graded mix. We did not cut the specimens to determine whether there was a variation between the outer and middle segments, but I would infer from the concordance of specific gravity results that we obtained a very uniform specimen by this method of molding. **Mr. Ulman.**

MR. PRÉVOST HUBBARD.—So far we have made no specific gravity determinations on different parts of our test specimens but one of the reasons why we have adopted the 1-in. high specimen in preference to the 2-in. high specimen is the difference in compression that we were sure we were getting throughout the depth of material. Such difference in compression was apparent to the eye and that it existed was shown by the fact that more variable results between different specimens of the same batch were obtained with the 2-in. high briquettes than with the 1-in. high briquettes. **Mr. Hubbard.**

I think there is probably some difference in compression in even the 1-in. briquette. I do not believe it is possible in any way to produce a specimen which will be absolutely uniform in density throughout its entire bulk. I do not believe we are obtaining absolutely uniform density on the road by any possible means of compression.

Mr. Hubbard. I think, however, that in our test, by reducing the height of specimen to 1 in. we are getting near enough the same compression throughout to serve all practical purposes.

In connection with the methods of applying load which Mr. Perkins discussed, we have had that factor under consideration. I think there is a great deal in what he has said. We considered adopting some sort of impact test, and then for reasons which he mentioned we decided not to. But I want to call attention to the fact that the test which Mr. Field and I have described does not utilize a true static load. The load is not, however, suddenly applied in the strict sense of the word, but it comes nearer being a suddenly applied load than a static load, because while the load starts at zero, it builds up very rapidly to a maximum. Not more than two or three seconds is ordinarily required to build up the load from zero to a maximum, so that we do not have a static load such as existed in Mr. Milburn's test.

The same would be true of Mr. Anderton's test, except that he applied his load very slowly and takes measurements at different intervals. We started out on that basis, also attempting to measure the rate of flow, but we found we could get more logical results by applying our load just as suddenly as possible and registering the maximum, which once reached, was never reached again if the test was continued.

Mr. Perkins. MR. PERKINS.—One reason why I think it is important in tests of this character to keep the plunger in contact with the sample and then increase and decrease the load at rapid intervals, is the effect Mr. A. E. Schutte of our company obtained a few years ago in a test which unfortunately was not carried out in more than a preliminary way. A gas-filled rubber ball, such as small children play with, was placed on a sample of sheet asphalt pavement made in accordance with Richardson's "Ideal Mixture." Pressure of about 5 lb. was applied to the top of the ball suddenly, and equally as suddenly released, this pressure being sufficient to compress the ball vertically to about two-thirds its original diameter with corresponding increase in its horizontal diameter. The test was carried out at room temperature. After a few thousand repetitions of this pressure, the sample showed but a very slight depression directly under the ball, but did show a ridge around the ball about $\frac{3}{8}$ in. in height. Other mixtures were tried with similar results, but varying in degree. To ascertain what a given mixture will do under a given stress, we must duplicate that stress when making the test.

MR. SMITH.—In connection with the question of uniform compression of the test samples, our experience has been that better results were obtained under certain conditions by compressing the samples themselves with a plunger of less area than the surface being compressed. Bituminous surfaces when laid upon the street are compressed by the action of a roller which involves a kneading action and a rearrangement of particles. If the upper surface of the test specimen is acted upon by a plunger, say one-third the area of the top surface of that specimen, you will get a kneading action and a rearrangement of particles—a compression immediately underneath the plunger and a flow resulting in a raised portion of the surface adjacent thereto. **Mr. Smith.**

If, then, the position of the plunger is moved so as to overlap the first position and to cover partially a position which had previously been untamped, that same action will be repeated and if that movement is continued so that it eventually covers the entire surface, you will have obtained a kneading action by such methods which you can not get by the direct action of a full surface area plunger. We have also found that a final compression with a plunger having the full area of the exposed surface finishes up the specimen and gives us more uniform density throughout than we have ever been able to obtain by direct compression with a full area plunger, except perhaps in those cases in which a total pressure has been applied which was sufficient to result in a very considerable amount of fracture of the grains composing the mixture itself, which, of course, is undesirable.

MR. T. A. FITCH.[1]—We have conducted more or less sporadically for a period of years, tests attempting to evaluate bituminous pavements for their resistance to displacement and the papers just presented certainly have been interesting along that line. I simply wish to bring out that in all of the work we have done, from the very start the attempt has been made to correlate the synthetic specimens made in the laboratory with other specimens cut from actual pavements which have been under traffic for some time. **Mr. Fitch.**

This has been possible in our case by the use of a rather large disk, 5 or 6 in. in diameter, for the synthetic specimens, so that a similar piece could readily be cut from the streets for comparison, and in general the displacement results, which we hope to eventually incorporate in specifications to replace the present evaluation of ingredients, do parallel the conditions found in actual street traffic specimens.

[1] Testing Engineer, City Engineering Department, Los Angeles.

QUALITY OF OIL FOR SURFACE OILING OF EARTH ROADS AND STREETS

By F. L. Sperry[1]

Synopsis

This paper describes a series of service tests having for their object the determination of the suitability of various types of commercial petroleum road oils for the purpose of surface oiling earth roads and streets. The tests were conducted on six experimental oiled earth roads located in various parts of Illinois so as to represent a wide variety of soil, traffic and climatic conditions. These experiments were started in the spring of 1923 and were completed in April, 1925.

The road oils used in the tests varied in character from practically pure paraffin-base oil to pure asphaltic-base oil made by different refining processes.

The outstanding conclusion drawn from these experiments is that certain types of cracked and paraffin-base oils that have been traditionally considered as unsuited or inferior for road oiling purposes are well suited for use on earth roads and streets.

For most road purposes, binding qualities and the properties of adhesiveness and cementitiousness are regarded as the determining factors in judging the quality of a road oil, but for an oil intended for earth roads, experience in Illinois and the experiments described in this paper indicate that binding qualities are of secondary importance and that resistance to emulsification and waterproofing qualities are the more important. In Illinois there are at the present time upwards of ten thousand miles of oiled earth roads and streets requiring annually approximately forty-five million gallons of road oil. For a number of years all of this oil has been purchased under specifications and tested for compliance therewith, and this work in connection with the experimental work done has afforded an unusual opportunity to observe the results of various types of asphaltic, semi-asphaltic and paraffin-base oils on earth roads.

When oil was first used on earth roads in Illinois, it was naturally assumed that asphaltic-base oils would be superior to semi-asphaltic or paraffin-base oils, but experience gained over a period of years on a large mileage of roads has not borne out this assumption. On these experiments, every commercially available type of oil has been thoroughly tested, including oil ranging from almost pure paraffin base to

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Mexican pure asphalt-base oils and the paraffin and semi-asphaltic oils have consistently given the best results.

The earliest attempts at earth road oiling in Illinois were primarily a dust preventive measure. It was soon observed, however, that the oil produced waterproofing effects and at the present time oil is used not only for dust-laying purposes but with the object of forming a waterproof and dustless surface for all-season traffic. Throughout this discussion the quality of oils are judged upon their value as a material for forming an all-season wearing surface and not as dust-layers only.

DESCRIPTION OF EXPERIMENTAL ROADS

As a means of securing accurate data upon various types of oil as well as the means of studying other earth road oiling problems, the Illinois Division of Highways in the spring of 1923 constructed two experimental roads, one of which is located near Cambridge in the northern part of the State and the other at Rosemond in the south central portion. In the spring of 1924, four additional experimental oiled earth roads were constructed in various parts of the State and so located as to represent as wide a range of traffic, soil and climatic conditions as possible. Observations of each oil treatment continued over a period of one year, as this is considered the life of an oiled earth surface without re-treatment.

Each experimental project is approximately three miles in length and is divided into sections of about 1500 ft. In the six roads there are altogether 99 sections, each of which has received a different oil treatment or else has different soil or traffic conditions so there are no two sections alike. All of the roads are well-drained and well-graded earth roads located on both primary and secondary routes. The traffic by actual count over a period of one week varied from as low as 150 per day on one road to 1500 per day on another. All grading and other preparatory work upon the roads and the application of the oil was done under the author's supervision and in accordance with the Illionois specifications.

ANALYSIS OF OIL

In Table I are given the physical and chemical characteristics of the different oils employed in the experiments. Each analysis represents a commercial grade of oil being regularly sold in Illinois for earth road oiling purposes. It will be noted that a large variety of oils are included both in respect to the method of manufacture and type of crude used.

RESULTS OF SERVICE TESTS

Oils Nos. 1, 2 and 3 consist of straight run residuums. The low percentage of naphtha-insoluble bitumen indicates that they are practically pure paraffin-base oils. When applied to the road these oils penetrated well, or what is the same thing, they were evenly and completely absorbed by the soil. Under traffic they ironed out into an excellent wearing surface, resembling asphalt, on some sections, where conditions were favorable. These oils retained their life and waterproofing qualities well, and at the end of one year were in better

TABLE I.—PHYSICAL AND CHEMICAL CHARACTERISTICS OF ROAD OILS USED IN ILLINOIS EXPERIMENTS.

Serial No.	Nature of Oil	Grade of Oil	Specific Gravity at 25° C.	Engler Specific Viscosity at 60° C.	Solubility in Carbon Disulphide, per cent	Insoluble in 86° Baumé Paraffin Naphtha, per ct.	Flash Point, Open Cup, deg. Cent.	Loss at 163° C., 5 hours, per cent	Solid Residue of 100 Penetration, per cent	Ductility of Solid Residue at 25° C., cm.
1	Straight run residuum from paraffin petroleum	Light	0.933	13.7	99.9	1.4	190	0.7	56	11
2		Medium	0.939	21.4	99.9	2.9	210	0.7	66	13
3		Heavy	0.939	31.5	99.6	1.4	238	0.45	68	22
4	Semi-asphaltic residuum blended with pressure tar	Light	0.980	13.8	99.6	11.9	130	7.9	66	10
5		Medium	1.002	23.9	99.6	14.0	125	9.7	67	4
6		Heavy	1.013	30.1	98.8	16.4	134	4.5	69	5
7	Pressure tar from paraffin or semi-paraffin petroleum	Light	1.030	13.6	99.9	11.8	170	2.7	61	100+
8		Medium	1.045	18.9	99.9	12.4	185	2.0	68	100+
9		Heavy	1.056	32.3	99.9	14.2	195	1.6	72	100+
10	Cracked residuum from paraffin or semi-paraffin petroleum	Light	0.938	10.3	99.6	5.1	165	4.1	65	2
11		Medium	0.944	16.1	99.6	6.3	180	1.9	74	2
12	Paraffin residuum blended with Mexican asphaltic crude	Light	0.947	13.2	99.9	5.7	150	5.2	57	21
13		Medium	0.959	23.8	99.6	10.5	126	6.1	64	33
14	Mexican pure asphalt base oil	Medium	0.963	23.5	99.9	15.6	105	5.6	63	100+
15	Cut-back Mexican asphalt base oil	Light	0.961	15.6	99.8	14.9	98	15.2	59	87

condition than any of the other oils, except the pressure tar represented by samples Nos. 7, 8 and 9.

Oils Nos. 4, 5 and 6 are said to have been made by blending rather heavy straight-run residuum with enough pressure tar to adjust the viscosity to specification limits. Both the pressure tar and residuum used in the preparation of these oils were obtained from Southern Arkansas crude. These crudes, while classed as semi-asphaltic, rather closely approach the asphaltic in character. The high percentage of naphtha-insoluble bitumen would usually be interpreted as indicating good binding and adhesive qualities. The lack of ductility of the solid residues and the slightly low total bitumen are probably due to the oils having been overheated in refining. When applied

to the road these oils did not penetrate well. The soil seemed to exert a selective absorption, producing a fractionating effect, certain fractions, probably the lighter, being the more readily absorbed, while the heavier fractions tended to filter out and remain on the surface. This peculiarity of these oils proved a handicap in applying them and in some cases was the cause of a thin, non-uniform and pitted wearing surface. When applied during extremely hot weather and under otherwise favorable conditions, these oils would penetrate well and under such conditions gave good results for a limited period. They lost their life more quickly than most other oils and during the spring

FIG. 1.—A Section at Galesburg, Ill., Oiled with Paraffin-Base Oil (No. 2, Table I). This picture was taken March 11, just after the frost came out, and shows the oil to be in excellent condition.

following their application all of the sections treated with these oils had either failed entirely or were in poorer condition than adjoining sections oiled with other types of oil.

Oils Nos. 7, 8 and 9 consist of reduced Burton-process pressure tar. The peculiar tar-like characteristics of these oils are due to the process of manufacture rather than the type of crude used. The highly ductile and adhesive solid residues obtained by the evaporation of these oils would seem to indicate superior binding properties, but when applied to the road the reverse was found to be true. These oils show remarkable penetrating properties and absorbing power and consequently they tend to absorb more material than they have the power to hold in place, and this peculiarity no doubt accounts for the fact that they do not exhibit better binding qualities on the road.

These oils retained their life longer on the road than any other type of oil and under average soil conditions existing in Illinois, their penetrating qualities are of great advantage. With the exception of one case of dusting, these oils have given better results than any other type of oil used in the experiments.

Oils Nos. 10 and 11 consist of residuums from high-pressure cracking stills employing semi-asphaltic crudes. Only enough light material has been blended with the residuum to adjust the voscosity to specification limits. The solid residue yielded by these oils is of a short waxy or greasy nature without the remotest resemblance to asphalt. The analysis would certainly indicate an inferior product

FIG. 2.—Note the abrupt change in the condition of the road at the section juncture. The portion of road in the foreground was oiled with asphaltic-base oil (No. 5, Table I). Road in background was oiled with pressure tar (No. 8, Table I). By March 6, when this picture was taken, the asphaltic oil had so completely lost its life as to be entirely ineffective as a waterproofing medium. The pressure tar was still in good condition.

for road oiling purposes. When applied to the road, these oils penetrated fairly well. While there appeared to be a slight tendency toward fractionation there was not enough to be detrimental. The oils retained their life on the road exceedingly well and at the end of one year were in as good a condition as any of the oils used, with the possible exception of the pressure tar. These oils did not exhibit strong binding properties on the road and possibly in drier climates or with more friable soils would fail to function as dust preventives. Nevertheless they proved satisfactory under the conditions in which they were used.

Oils Nos. 12 and 13 consist of straight-run semi-asphaltic or paraffin residuums of about the consistency of fuel oil to which has been blended enough heavy Mexican oil to bring the viscosity up to specification requirements. These oils did not retain their life on the road as well as the straight paraffin and semi-asphaltic residuums nor could it be observed that the addition of the Mexican oil served to improve the binding or mat-forming qualities of the oil. At the end of one year these oils were in poorer condition than most of the other oils.

Oils Nos. 14 and 15 are of Mexican origin, No. 14 being a straight-run residuum and No. 15 a cut-back product. The analyses of these oils indicate they are of a pure asphalt base with good binding and adhesive qualities and would be regarded as high-class products. When applied to the road, the behavior of these oils was surprising. Within a few weeks after application, the oils had become so emulsified and had so completely lost their life that they were practically useless either as dust preventives or waterproofing mediums. About six weeks after application, a shower caused 2 in. of mud and as soon as the surface dried off it was as dusty as an unoiled road. In fact, the surface presented the appearance of a road which had never been oiled.

CONSISTENCY OF OIL FOR EARTH ROADS

Illinois specifications for semi-asphaltic oils for earth roads are as follows:

	E-2 Light	E-3 Medium	E-4 Heavy
Specific Gravity at 15.5° C	Not less than 0.910	Not less than 0.920	Not less than 0.930
Specific Viscosity (Engler) at 60° C	8.0 to 16.0	16.1 to 28.0	28.1 to 42.0
Total Bitumen, per cent	Not less than 99.5	Not less than 99.5	Not less than 99.5
Flash Point	Not less than 80° C.	Not less than 80° C.	Not less than 80° C.
Solubility in 86° Baumé Paraffin Naphtha	When specific gravity is under 0.970, not over 8.0 per cent	When specific gravity is under 0.975, not over 9.0 per cent	When specific gravity is under 0.980, not over 10.0 per cent

The specifications for asphaltic oils for earth roads are as follows:

	E-1 Light	E-5 Medium
Specific Gravity at 15.5° C	0.940 to 0.970	0.950 to 0.980
Specific Viscosity (Engler) at 60° C	8.0 to 16.0	16.1 to 28.0
Total Bitumen, per cent	Not less than 99.5	Not less than 99.5
Flash Point	Not less than 80° C.	Not less than 80° C.
Solubility in 86° Baumé Paraffin Naphtha, per cent	Not less than 11.0	Not less than 12.0
Ductility of Solid Residue	[a]	[a]

[a] When 50 g. of the oil are evaporated at 250–260° C. until the residue has a penetration at 25° C., 100 g., 5 seconds, of 90 to 100, the residue shall have a ductility at 25° C. of not less than 50 cm.

In these specifications three grades of oil are included, based upon their Engler specific viscosity taken at 60° C. This unusual temperature for taking the viscosity was selected for the reason that all three of these grades of oil are sufficiently fluid at 60° C. to be tested and give a good range of readings with the Engler viscosimeter. At the more or less standard temperatures of 40 or 50° C., the E-3 and E-4 grades are frequently not sufficiently fluid to be tested with the Engler apparatus, and at 100° C. all three grades are too fluid to give a satisfactory range of readings.

All three of these grades of oil have been used on the experimental roads with approximately equal success. In fact, at no time could any great difference be noted with the three different grades of oil. A few years ago, a grade of oil lighter than E-2, having a viscosity at 60° C. of less than 8 and a solid residue content between 40 and 55 per cent, was extensively used in Illinois, but it was found that frequently during dry periods and when applied on friable soils it was too light to be effective as a dust preventive and its use has been discontinued. Oils heavier than the E-4 grade have been tried to a limited extent but it was found that such oils will not penetrate well nor be absorbed by the soil; so it appears that the three grades of oil covered in the specifications include the full range of viscosity that can be used successfully for earth road oiling.

PENETRATING QUALITIES

In applying the different oils it was observed that some penetrated much better and were more quickly and completely absorbed than others and that the degree of penetration had much bearing upon the success of the oils on the road. Oils which penetrated poorly or were not completely absorbed made a thin skin-like mat which tended to pit under traffic and make a rough non-uniform wearing surface. The oils which penetrated best invariably developed the best wearing surface and gave the least trouble in applying.

It is not uncommon among specification writers to insert a clause requiring a minimum loss at 100 or 163° C. (212 or 325° F.), with the object of insuring a certain percentage of light volatile material which is intended to improve the penetrating qualities and later by its evaporation upon the road, develop the asphalt content and the mat-forming qualities of the oil. Oil No. 15 is of this sort, but when applied to the road no apparent advantage in penetrating qualities could be noted over other oils. On the other hand, pressure tar, which is entirely free from light oils, showed much better penetrating qualities than any other type of oil used. By reason of the practical nature of

these tests it was impossible to discern small variations in penetrating qualities, but it was apparent that homogeneous oils in general showed better penetrating qualities than blended products. With the latter oils a tendency toward filtration and selective absorption by the soil was noted, the lighter constituents tending to penetrate, leaving the heavier fractions unabsorbed on the surface. A cut-back asphaltic oil when applied to a gravel or similar type road may develop an asphaltic mat by the evaporation of its volatile fractions, but the same result cannot be accomplished on an earth road. On an earth road the oil is absorbed at once by the soil and if a mat is formed at all, it will be of an oil-earth composition and a paraffin or semi-asphaltic oil is just as effective in developing such a mat as any other kind of oil.

SOLID RESIDUE TEST

There is a tendency in the trade to grade road oils upon the basis of their solid residue or so-called "asphalt content," regardless of the purpose for which they are to be used. While it is not the purpose of the author to depreciate the value of this test for certain materials, these experiments indicate that it has very little use as a basis for grading or specifying semi-asphaltic or paraffin-base oils intended for earth road oiling purposes. The residues yielded by semi-asphaltic and paraffin oils are usually short and waxy and in many cases bear little resemblance to asphalt. Such oils do not evaporate to any extent after being applied to the road nor do they depend for their efficacy upon a residue of asphaltic binder, such as cut-back products are supposed to yield upon the evaporation of their lighter constituents subsequent to application.

The test for solid residue when applied to paraffin and semi-asphaltic products is tedious and susceptible to inaccuracies due to the length of time and high temperatures required for carrying out the evaporation. Oils of this type usually require from twenty to thirty hours for evaporation as compared with one to four hours for asphalt-base products.

While the test may have some value from the standpoint of the chemist as a means of establishing the identity and character of new materials, its only value as a control test is as a rough indication of the consistency of an oil, a quality much better expressed by the standard specific viscosity test. For these reasons it is the author's belief that the solid residue or asphalt requirement should never be used as a control test or as a basis of purchase or specifications for paraffin or semi-asphaltic oils intended for earth road oiling purposes.

CONCLUSIONS

1. Semi-asphaltic and paraffin-base oils are superior to asphaltic-base oils for use on earth roads for the reason that they resist emulsification and retain their life much longer in contact with the soil and moisture.

2. Reduced pressure tar is a satisfactory oil for earth roads.

3. Binding qualities and adhesiveness are secondary in importance to the ability of the oil to resist emulsification and retain its life in contact with the soil. This may not hold true in extremely dry climates.

4. Cut-back or mat-forming oils which develop their asphalt content by the evaporation of their lighter constituents after their application are not well suited for earth roads.

5. Non-volatile and homogeneous oils penetrate better and are more evenly and uniformly absorbed by the soil than blended products. Blended oils tend to separate or fractionate when applied.

6. The so-called "asphalt or soljd residue test" is unsatisfactory as a control test or as a basis of purchase or specifications for semi-asphaltic or paraffin oils for earth roads. The test may be of some value from the standpoint of the chemist as a means of establishing the identity and characteristics of unknown products, but beyond this is of no value.

It should be borne in mind that the foregoing conclusions are based on tests made on specific soils in Illinois and that different soil or climatic conditions might lead to different conclusions. It is also important that these conclusions be not confused with the surface treating of gravel or macadam roads, which present entirely different problems and for which entirely different materials may be denoted.

Acknowledgments.—The writer is indebted to the following county road officials for assistance in carrying out the experiments: J. H. Reed, County Superintendent of Highways, Cambridge, Ill.; Elmer Mitchell, County Superintendent of Highways, Delavan, Ill.; E. E. Vincent, Highway Commissioner, Rosemond, Ill.; A. B. Hurd, County Superintendent of Highways, El Paso, Ill.; Boyd Dappert, County Superintendent of Highways, Taylorville, Ill.; William Querfeld, County Superintendent of Highways, Clinton, Ill.; H. M. Roy, County Superintendent of Highways, Galesburg, Ill.

MACHINE FOR THE DETERMINATION OF THE PLIABILITY OF PREPARED ROOFING AND THE BREAKING POINT OF BITUMEN

By Charles S. Reeve[1] and Frank W. Yeager[2]

Synopsis

This paper describes a machine which is believed to offer a marked improvement over previous types of apparatus for the tests noted. As applied to roofing it operates on the principle that when a piece of roofing is bent at a given temperature over a $\frac{3}{16}$-in. mandril until cracks appear in the coating, the angle through which the piece is bent is a measure of the pliability of the product at that temperature. The apparatus is of simple construction and adapted to operation in a small constant-temperature bath. The rate of bending is easily controlled. Results are presented to show the range of reproducibility for the same and different operators.

The application of the apparatus to the determination of the breaking point of bitumen is also developed and shown to be a great improvement over the machine described by Lloyd and Sharples[3] without any change of values.

The degree of pliability of a prepared roofing has long been considered an important index of its quality, pliability being defined as that property which permits bending to a greater or less degree without developing breaks in the asphaltic coating. Pliability without undue limpness is desirable as it permits the unrolling, handling, and placing of the sheet, shaping over eaves, hips, deck edges, etc., without cracking the coating. A roofing with a coating that cracks or breaks under the above conditions is at a distinct disadvantage, as those points where the coating no longer forms a continuous membrane have decreased resistance to the destructive agencies of the weather to which all roofs are subjected and which they must withstand for long periods to give satisfactory service.

For the proper fabrication of roofing, a simple and accurate means for determining its pliability is therefore of considerable importance. Simple rough bending tests have usually been resorted to for determining this characteristic, and a method has been standardized to some extent by Herbert Abraham. This latter method consists in bending a sample of the roofing at a predetermined temperature

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[2] Chemist, The Barrett Co., Grantwood, N. J.

[3] H. E. Lloyd and P. P. Sharples, "Apparatus for Determination of Breaking Point of Pitches," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part II, p. 782 (1919).

over a series of mandrils of gradually decreasing diameter, rating the roofing by the number of the mandril over which the sample first shows a surface break.[1] While the introduction of this method was a step forward toward standardization of a suitable test, the method is entirely without mechanical control, and the bending, performed directly by the hands of the operator, permits a wide play of personal equation. To meet the need for a more closely controlled and reproducible test, the method and apparatus here described was developed.

The idea underlying the method is that when a piece of roofing is bent over a mandril of fixed diameter until cracks appear in the coating, the angle through which the piece is bent is a measure of the pliability of the roofing, that is, the greater the angle of bending permitted without cracking, the more pliable the roofing.

In embodying this idea in a practical mechanism, various means of holding and bending the test piece were tried, with the result that the very simple and effective arrangement shown in Fig. 1 was evolved. The essential parts of the apparatus are: the clamp, *A*, holding the strip of roofing under test, and the large wheel, *D*, carrying the pin, *C*, one inch distant from its center and projecting perpendicularly from its surface. The clamp, *A*, is designed to hold the lower 2 in. of a roofing specimen 4 in. long and 1 in. wide. This clamp is located directly in front of the large wheel so that the back of the clamp, the top of which is rounded to a $\frac{3}{16}$-in. radius, serves as a mandril located on the central line of the bearing on which the large wheel revolves. Thus placed, the pin, *C*, projecting perpendicularly from the face of the large wheel, and the rounded back of the clamp, are parallel.

When a strip of roofing, *B*, is held in the clamp, as shown in Fig. 1, rotation of the large wheel brings the pin into contact with that part of the strip projecting above the clamp, and by continuing the rotation, the pin, bearing on the strip, causes it to bend about the $\frac{3}{16}$-in. rounded back of the clamp. The limit of the rotation is reached when the strip has been bent double over the back of the clamp, practically equivalent to 180 deg. of bend.

The rotation of the large wheel and the consequent bending is readily accomplished and controlled by turning the handle, *E*, operating the small pinion gear in mesh with the large wheel. The rate of turning is controlled by a metronome to a speed equivalent to bending the specimen through 180 deg. in 30 seconds.

For convenience in measuring the angle through which the strip is bent, the protractor, *G*, graduated in degrees, is fixed to the wheel, *D*, in a suitable position. In starting a test the pin is brought in contact

[1] H. Abraham, "Asphalts and Allied Substances," p. 560.

with the projecting strip of roofing when it is in a vertical position and the pointer shown just below the set screw, F, is adjusted to indicate zero. This feature provides for roofings of different thicknesses requiring slightly different initial positions of the large wheel.

As will be seen from Fig. 1, the upper surface of the strip at the point where the bending occurs is at all times clearly visible, so that the development of a break in the coating is readily discerned. When

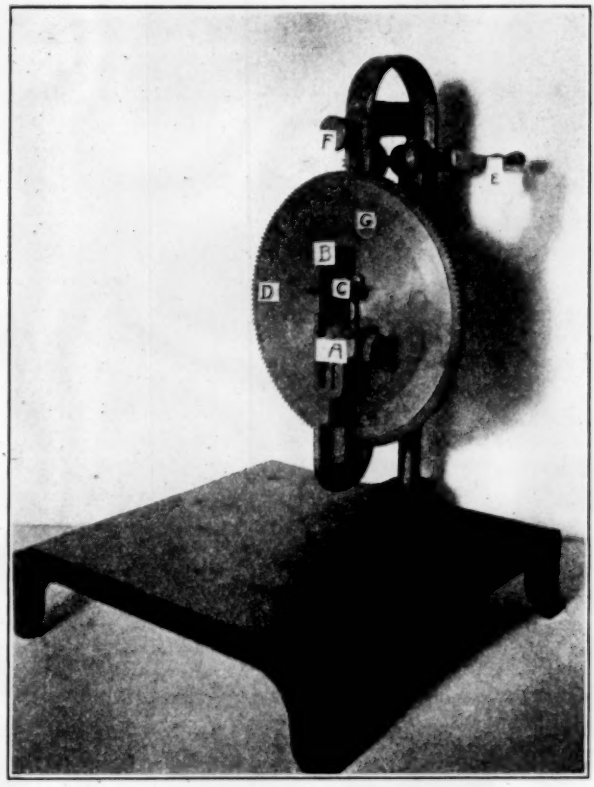


FIG. 1.—Apparatus for Measuring the Pliability of Roofing.

this occurs, the motion of the machine is stopped and the angle of bending read by the indicator and protractor. This value is recorded as a measure of the pliability of the roofing.

To secure reproducible test conditions, the necessary temperature control is obtained by means of a water bath. The large wheel and clamp are carried on a vertical support so designed that they can be lowered into the water bath which is placed on the broad base of the machine and which is made high enough to provide a water level just

above the upper end of the test specimen. Ordinarily, tests are run at a temperature of 50° F., representing moderately cool weather, although, if desired, any temperature obtainable by water or brine baths may be used. All parts of the machine are of brass and the protractor of nickel silver, so that no rusting takes place.

The clamp is of simple construction and in its design particular attention was given to ease of operation so that test pieces can be readily inserted and removed without raising the apparatus from the control bath. In order to secure a high degree of visibility an electric lamp with a suitable reflecting arrangement may be used to throw a strong light on the bending surface of the test piece, thus enabling a sharper determination of the end point.

The following figures are results obtained on two roofing samples of different degrees of pliability.

TABLE I.—PLIABILITY VALUES OBTAINED ON TWO SAMPLES OF ROOFING.

	Sample No. 1		Sample No. 2	
	Operator A	Operator B	Operator A	Operator B
Pliability	55	70[a]	135	153
	51	45	110[a]	165[a]
	46	58	120	125
	46	60	135	145
	38[a]	50	130	139
	50	58	105[a]	125
	..	45	...	...
	..	50	...	...
Average	50	52	130	137

[a] Omitted from Average.

The pliability of a roofing is to some extent dependent upon the so-called breaking point of the asphalt used in the coating. This is commonly defined as the maximum temperature at which a standard test specimen will break in two pieces when subjected to a bending stress. A method for determining this characteristic has been described by Lloyd and Sharples.[1] The apparatus described by them gives accurate and easily reproducible results, but the designers stated that it was capable of improvement in the way of overcoming certain inconveniences of operation and to meet the principal criticism that the specimen was not subject to observation during the test. The application of the above described machine to this test therefore suggested itself.

If, in place of the roofing specimen, a standard test piece of bituminous material, $\frac{1}{4}$ in. thick, 1 in. wide and 4 in. long (the same

[1] H. E. Lloyd and P. P. Sharples, "Apparatus for Determination of Breaking Point of Pitches," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part II, p. 782 (1919).

width and thickness as used in the earlier machine), is inserted in the clamp, the test piece may be subjected to a uniformly applied bending stress. If the material is flexible enough at the temperature of the test it will bend without breaking, or, if the test is conducted at a sufficiently low temperature, the specimen will be rigid enough to break instead of bending. The apparatus permits of having the test piece under direct observation at all times, and the break, which

TABLE II.

	Breaking Points, deg. Cent.	
	New Machine	Lloyd and Sharples Machine
Asphalt No. 1	14	15
Pitch No. 1	14	14
Pitch No. 2	−1	−2

usually extends entirely through the specimen, is easily detected. In addition, the test pieces are readily inserted and removed from the apparatus, thus meeting the two main objections to the earlier machine.

As with the earlier machine, bendings are repeated at higher or lower temperatures as the case may be until a difference of 1° C. separates a break from a no-break test; the lower temperature is then reported as the breaking point of the bituminous material. In conducting this test the machine is operated by metronome control to give 180 deg. of bend in 10 seconds.

It is of particular interest to note that the results obtained with the new machine are practically identical with those obtained by the older apparatus, as is evident from Table II.

DISCUSSION

Mr. Adams. MR. W. C. ADAMS[1] (*by letter*).—Shortly after Lloyd and Sharples' paper on the breaking temperature of pitches was published in 1919,[2] we adopted their method for testing the pliability of asphalt, tar, bituminous putty, and bituminous roofing, by determining the temperature at which the material does not crack, and the temperature at which it breaks in two. Our contrivance consisted of a hinge similar to that of Lloyd and Sharples, made of two strips of wood set in a narrow tank to avoid the work of changing the temperature of a large volume of water. To bend the test strip around the 5-mm. rod, a string from each end of the hinge was run over a bar above the center of the hinge, and the ends attached to our ductility machine. Our results were satisfactory, but the time required, about fifteen minutes, was excessive. We then tried pulling the hinges together by hand in about two seconds, that is, at a conveniently fast rate.

This gives breaking temperatures a little higher for felts than by the slow method and 3 to 5° F. higher for pure bitumens. We find the difference between breaking in two and not cracking is 4 to 6° F., and check tests come within 2° F., which for practical work is satisfactory.

I consider the method far superior to the use of different sized mandrils, and that measuring the angle of bending that caused breaking at a standard temperature should only be used as a guide to the breaking temperature, or possibly for routine tests on one type of material.

The determination of the breaking temperature gives far more valuable information than does the ductility test at 77° F. What a bitumen does at 77° F. is not of much importance in judging what it will do when cold and when hot, which is of vital importance in judging of the fitness of bitumens. For this reason, I think the determination of the breaking temperature should be substituted for the ductility test, or at least given priority, for pure bitumens and putties. For bituminized felts it is necessary.

The machine developed by Reeve and Yeager is a great improvement in appearance over the older forms and adds some conveniences;

[1] Consulting Paving Engineer, Milton Hersey Co., Ltd., Montreal, Canada.

[2] H. E. Lloyd and P. P. Sharples, "Apparatus for Determination of Breaking Point of Pitches," *Proceedings*, Am. Soc. Testing Mats., Vol. XIX, Part II, p. 782, (1919).

but the hinge form, folded by hand, gives such satisfactory results that I think no one need defer using this method till the new form is obtainable. **Mr. Adams.**

MESSRS. C. S. REEVE AND F. W. YEAGER (*authors' closure by letter*).—We appreciate Mr. Adams' criticisms and suggestions, but one should not lose sight of the dual purpose served by this apparatus, that is, determining the temperature at which a bituminous material will break under standard conditions and determining the angle through which a material may be bent under standard conditions without breaking. Either use of the apparatus may be employed to advantage in accordance with the nature of the information desired. We quite agree that Mr. Adams will obtain reasonably satisfactory breaking point results by the simplified method he described, and his results probably will accord fairly well with those obtained by our apparatus. However, control of testing conditions to the greatest extent possible is particularly necessary with bituminous materials, since the results obtained are usually dependent on arbitrarily adopted conditions. Hand operation, aside from the inconvenience in this test of working in brine at times below 0° C., is always subject to the personal equation, and it is for this reason that we believe a mechanically operated test for breaking point is as desirable as the similarly operated test for ductility. **Messrs. Reeve and Yeager.**

TESTS FOR HARDNESS, GLOSS, COLOR AND LEVELING OF VARNISHES

By A. H. Pfund[1]

Synopsis

This paper describes instruments for testing hardness, gloss, color and leveling of varnishes, as follows:

Hardness.—This property is tested by means of a modified Brinell test. A small sphere about 2 mm. in diameter is forced into the varnish film. The diameter of the circle of contact between varnish and ball is determined *while the two are in contact.* Measurements are taken with a micrometer eyepiece and a microscope supplied with a special illuminating device. The scale of hardness is made proportional to the load necessary to produce a circle of constant diameter. Varnishes of decreasing oil-length reveal an increasing hardness.

Gloss.—It is found necessary to cover the great range of glosses on two scales:

1. Normal gloss scale: This scale covers the range from highest glosses to "flat" surfaces. The instrument measures the intensity of the specularly reflected light. It is particularly applicable to the study of loss of gloss due to weathering.

2. High gloss scale, covering the narrow range of variations exhibited by surfaces of very high luster.

Color.—Because of the fact that colored glasses, as a class, are more nearly permanent than colored solutions, as a class, a color comparator, involving the use of a wedge of amber glass, was developed. In order to cover the wide range in color and transparency displayed by varnishes it was found necessary to use also a hollow wedge, to be filled with varnish, in conjunction with the amber wedge. The two wedges are reversed with respect to one another and are mounted on the same slide behind a fixed vertical slit. Readings are taken by moving the slide until the colors of the two wedges, as seen through the narrow slit, are the same. The Gardner-Holdt color-scale is retained.

Leveling.—The instrument gives information concerning the time interval which may elapse between the application of a varnish and the development of brush marks. So as to simulate the rapid motion of a brush, an arm, bearing three weighted needles whose points pass through the varnish film to the underlying glass plate, is swept across the surface rapidly. The plate is then displaced and at intervals of 5 minutes, the rapid motion of the arm is reversed. Failure to level is detected by noting the appearance of faint undulations on the surface.

A test for a given physical property ought to be so devised that but a single physical quantity is measured and that the result can be expressed in absolute c. g. s. units. This, unfortunately, is not always

[1] Associate Professor of Physics, Johns Hopkins University, Baltimore, Md.

possible. Frequently, tests for such important properties as hardness, toughness, resiliency, etc., are desired in spite of the circumstance that no rigorous physical definitions for these quantities exist. Then too, conditions of usage impose a test which produces a simultaneous variation of a number of physical properties. Measurements of this type are usually expressed in terms of an arbitrary scale whose increasing integers do not necessarily bear a linear relation to the quantities undergoing variation. The only excuse for the existence of such types of measurement is that they alone yield the desired information.

The above considerations apply to the numerous tests for varnishes. At the present time there is no general agreement in the varnish industry as to the character of the tests—hence, an intercomparison of varnishes from the numerical data on hand is not

FIG. 1.

possible. It is the purpose of the present paper to take at least a step toward the desired goal. The whole problem is so large and so difficult that its complete solution will be reached only as a result of cooperative effort. In the development of the tests, it was sought to find unambiguous, sharply defined criteria, and also, to eliminate, as much as possible, the element of personal judgment. Decision as to the ultimate scales for hardness, gloss, etc., is left for the future.

HARDNESS OF VARNISHES

After numerous, unsuccessful attempts to grade the hardness of varnishes by means of the scratch-test—in which graded pencils, crystals, etc., were used—a modified Brinell test was developed. At first a steel ball bearing, $\frac{1}{16}$ in. in diameter, was forced, under load, into the varnish films and the diameter of the resultant circular impression was measured under the microscope. However, due to the elasticity of the varnish, the marks would frequently shrink and almost fade away before measurements could be taken. In the hope of remedying this defect by making a more incisive mark, an artificial "thumb nail" was developed. The earlier form consisted of a steel

ring about 1 cm. in diameter and turned down to a thickness of about 0.003 in. By mounting this ring on the end of a weighted bar and inclining the edge of the ring slightly with respect to the varnished surface, a curved mark or cut similar to that made by the thumb-nail was left on the surface. Because of sensitiveness to slight variations in the angle of inclination, the tool was modified at the suggestion of Mr. D. V. Gregory, as shown in Fig. 1. A steel shim, *L*, 0.0035 in. thick is cut or ground off at one edge into a circular arc having any desired radius from several feet to a fractional part of an inch. The edge is rounded off with fine emery paper and the blade is soldered to a vertical post *P* set in the end of a weighted bar.

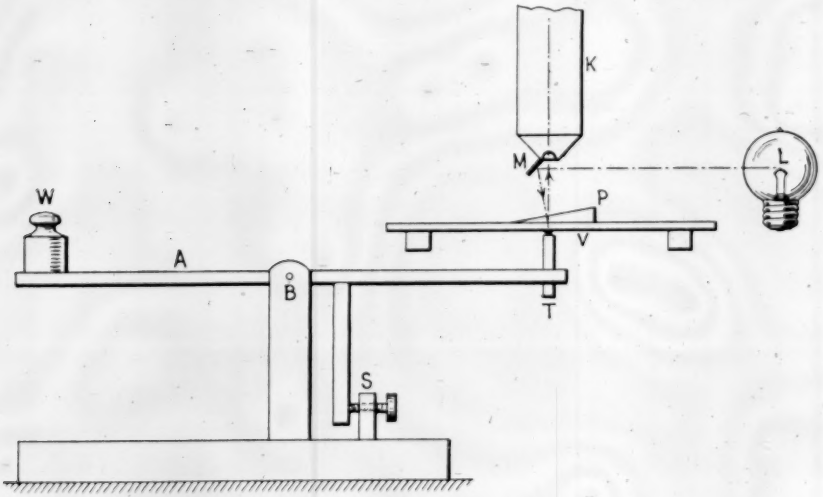


FIG. 2.—Apparatus for Testing Hardness of Varnish.

Since the rounded edge meets the varnish surface perpendicularly, no special care in mounting is required. Clearly, the harder the varnish, the shorter is the mark left by the cutting edge. The measurements obtained with this apparatus closely checked the findings of practical varnish men.

While this method yielded permanent marks when hard or even medium soft varnishes were studied, it was found that for very soft, elastic varnishes, the marks were more or less transient in character. While the method to be described overcame this difficulty entirely, it is to be noted that the new method of illumination may also be applied to the present tool with the curved edge.

Briefly, the new method consists in measuring the diameter of the circle of contact between the varnish and a small sphere while

the two are still in contact. The arrangement is shown in Fig. 2. An arm, A, about 20 cm. long is pivoted centrally on a horizontal axis, B, and is loaded by a weight, W. The other end of the bar bears a brass post, T, in whose upper end a steel or glass ball, about 2 mm. in diameter, is embedded. This ball is then forced into the varnish film V on the lower side of the glass plate, resting on the stage of a microscope from which all mirrors, condensers, etc., have been removed. By means of the screw S the ball is allowed to come into contact with the varnish film gradually. Illumination is furnished by an incandescent lamp L whose rays are reflected downward by the mirror M to the varnish film and then upward into the low-power microscope K. So as to eliminate the undesirable reflection from the upper air-glass surface, a 7-deg. glass prism is placed at P with a film of water between prism and plate. Under these conditions, the light reflected from the varnish-air surface fills the field of view of the microscope. A black circle with sharply defined edges reveals the area of contact between ball and varnish. The diameter of this circle is measured with a micrometer eyepiece.

Varnish films are prepared by the whirling disk method of Walker and Thompson.[1] After the films are hard-dry, the hardness may be determined roughly by keeping the weight W constant and measuring the diameter of the mark. This procedure is rapid but comparatively insensitive. A much better procedure is to calculate first the diameter of the mark resulting from a penetration of the ball through one-half the thickness of the varnish film,[2] then, to apply, successively, weights W_1 and W_2 yielding marks slightly smaller and larger, respectively, than the calculated diameter. By simple interpolation, the weight W_c necessary to produce a mark of the diameter calculated is determined. In all cases, the load should be applied for one minute before taking measurements.

A typical series of results, obtained for several varnishes with an improvised glass ball 2.1 mm. in diameter, is shown in the following table. The mass M_c tabulated is that required to produce a circular mark of the same diameter in all cases.

VARNISH	MASS, M_c	REMARKS
40 gal. Kauri, linseed-tung oil 1:1	3 g.	soft
20 gal. Kauri, linseed	10 "	medium
5 gal. Kauri, linseed	176 "	hard
Shellac	300 "	very hard

[1] P. H. Walker and J. G. Thompson, "Some Physical Properties of Paints." *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 465 (1922).

[2] Assuming the thickness t of the varnish film to be 0.001 in. (0.025 mm.) the diameter D of the desired mark is calculated from the formula $D = \sqrt{4\,\rho\,t}$, where ρ is the radius of curvature of the ball.

Assuming, temporarily, that a hardness scale proportional to the values of M_e is established, it is evident that the dispersion of the results is in the ratio of 1:100. The accuracy in the determination of M_e is at least as good as 5 per cent for any given varnish.

While results obtained by this ball method will, in general, parallel the findings of the practical varnish man who uses the thumb-nail test, contradictions are found for films which are easily indented by the ball but are so tough that they resist marring or penetration by the finger nail. In other words, the practical man has no way of distinguishing between hardness (resistance to indentation) and toughness. It is therefore essential that hardness measurements be

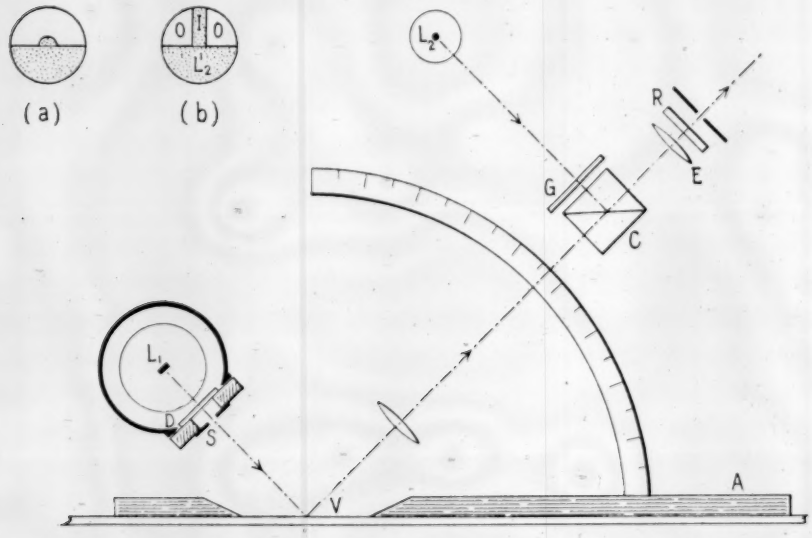


FIG. 3.—Apparatus for Testing Gloss of Varnish.

supplemented by stress-strain measurements of the type carried out by Nelson.[1] It is needless to add that reliable data can be obtained only by holding the varnish film at a fixed temperature and humidity, both before testing and during the time of test.

Upon dragging the varnished surface over the weighted ball at a small velocity, it is found that various varnishes behave quite differently. It is always possible to find a load such that the varnish surface is marred permanently.

[1] H. A. Nelson, "Stress-Strain Measurements on Films of Drying Oils, Paints and Varnishes," *Proceedings*, Am. Soc. Testing Mats., Vol. 21, p. 1111 (1921).

GLOSS

Extensive study has revealed the necessity for two gloss scales:

1. The "Normal Gloss Scale" covering the range from high gloss to "flat" surfaces;
2. The "High Gloss Scale" covering only the limited range of highest glosses.

Measurements on both scales are obtainable by means of the same apparatus. The general arrangement is shown in Fig. 3. The substantial metal plate A is laid upon the flat varnish panel V to be tested. Light from a 27-c-p. (6-8-volt) lamp L_1 passes, first, through a disk of diffusing glass D, then, through a circular opening, S, 3 mm. in diameter, and falls upon the varnish surface from which reflection takes place. The light is received by a lens which focuses an image of S upon the photometer cube C. A second small incandescent lamp L_2 illuminates a disk of opal glass, G, and this fills the other half of the photometric field. Viewing is accomplished by means of the eyepiece lens E and a red-glass filter, R. The appearance of the field of view is shown in the insert Fig. 3 (a). Measurements are obtained by displacing the lamp L_2 until the line of demarcation in the photometric field disappears. So as to give the widest range to the instrument, the angles of incidence and reflection can be varied from 15 to 75 deg. Differences between very glossy surfaces can be detected most readily at small angles of incidence. On the other hand, differences between very dull surfaces are most apparent at large angles of incidence. In extreme cases, it is therefore necessary to specify the angles of incidence at which measurements are made. Ordinarily, however, the angle is chosen at 45 deg.

According to the first method here proposed, gloss is defined as the ratio of the intensity of light reflected specularly from a varnish surface of 45 deg. incidence *to* the intensity of light, reflected under similar conditions from a perfectly smooth flat surface of the same material. Without going into details it is evident that a perfectly smooth, flat surface of a given varnish is not always procurable. In its place, the free surface of c. p. castor oil ($n = 1.480$) has been chosen. The castor oil is poured into a shallow dish painted jet black inside and is mounted in position at V for measurement.

Because of the wide range covered by this gloss scale, it is particularly applicable to the study of the weathering of surfaces. As for the use of the red glass, it may be said that it eliminates color differences and thus makes photometric settings more exact. Evidently, the color and brightness of the background (enamels) will

influence the values of gloss to some extent. It will be shown in a later paper how this error may be corrected for.

An alternative method for measuring gloss is based on the observation that the edges of images, reflected in a varnish surface, become increasingly diffuse as the gloss becomes poorer. If, then, two parallel wires, separated by a small distance, be viewed by reflection, with the sky as a background, the wires can scarcely be recognized at all if the gloss is only moderate and the angle of incidence is nearly normal. However, by increasing the angle of incidence more and more, the wires are ultimately seen distinctly double. This criterion of "resolving-power" was incorporated by the author in a gloss-meter which was constructed five or six years ago. Because of the circumstance that the instrument was incapable of differentiating between surfaces of very high gloss, no description appeared in print. Now that a separate scale for high gloss is proposed, the older instrument is again brought forward. The mode of operation to be followed in connection with the apparatus shown in Fig. 3 would then be to stretch two parallel wires 0.3 mm. in diameter and 0.3 mm. apart over the aperture S and to vary the angle of incidence until the wires can be recognized as distinctly double. This angle is termed the "image angle" and is to be recorded. Next, the wires are removed and the intensity of the specularly reflected light at this same angle of incidence is measured by means of lamp L_2. The results obtained by this procedure are but little influenced by the color and brightness of the background (enamels).

As stated, both of these arrangements are lacking in sensitiveness upon being used for the testing of "high glosses." A new criterion for the estimation of this property is based on the observation that if sky light, passing through a window, be reflected almost normally into the eye, it is found that, when the gloss is perfect, the vertical and horizontal window bars appear black. If, however, the gloss is imperfect, more or less light is reflected diffusely into the region of the window bars. The instrument shown in Fig. 3 is readily modified to incorporate the new criterion; the circular aperture S is replaced by a metallic strip 3 mm. wide—thus forming an artificial window bar—and the diffusing glass D is replaced by another transmitting much more light. The appearance of the photometric field is shown in the insert Fig. 3 (*b*) where the image of the strip I, *at its center*, is matched photometrically against the field L'_2. Defining, now, gloss as the ratio of light intensity outside the bar (field O) *to* the intensity inside the bar (field I) it is only necessary to displace the lamp L_2 until the fields O and I are, successively, in photometric balance.

On this scale, the numerical values for gloss lie between + infinity for perfect gloss *and* +1 for complete absence of gloss.

Since tests for gloss of varnishes are usually carried out by flowing the varnish over a black undercoating, the effect of the character of the background is eliminated. Whenever colored enamels are studied, corrections for the brightness of the background can be worked out readily.

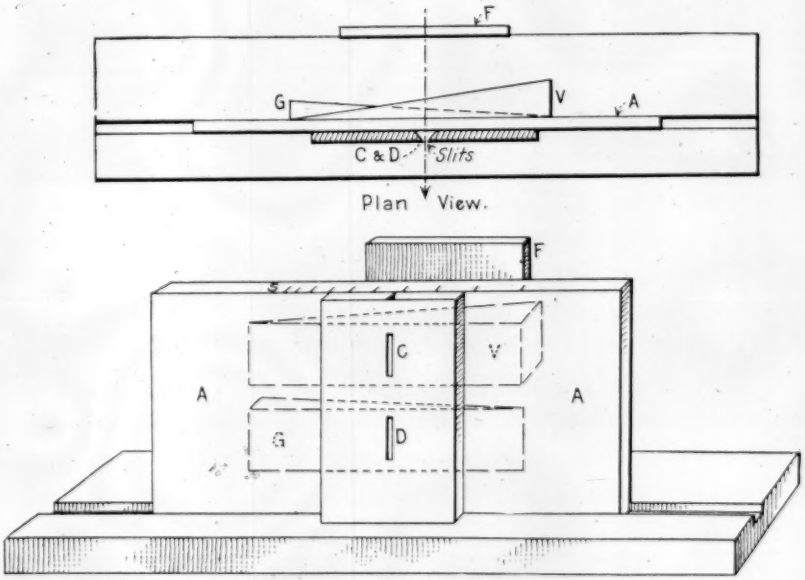


FIG. 4.—Apparatus for Determining Color of Varnish.

COLOR OF VARNISHES

Experience has demonstrated that tinted glasses, as a class, retain their color characteristics to a higher degree of constancy over a period of years than do colored solutions, as a class. Wide variations between different sets of fluid color standards for varnishes having been found,[1] it seemed worth while to attempt the use of amber glass (colored with carbon) in place of solutions. For preliminary attempts, a wedge of amber glass was used to match the color of a varnish layer of constant thickness. It was soon found, however, that, in order to match all varnishes,—ranging from the

[1] It is not improbable that the cause of these differences lies in the cork used to close one end of the color tubes. Similar color tubes, sealed off hermetically in a blow pipe, were made by the author several months ago and, thus far, have shown no change in color.

lightest yellow to the darkest red,—a wedge at least 3 ft. long and several inches thick at its base, would be required.

This difficulty was overcome by resorting to the construction shown in Fig. 4, in which *A* is a metal plate with two horizontal slots covered over by a wedge of amber glass *G* and a hollow-glass wedge *V* filled with varnish. Sky light falls upon a piece of diffusing glass *F* which in turn sends light through the two wedges and through the slits *C* and *D*, 1 mm. in width. By means of a simple viewing device, the light from the two slits is spread out so as to fill the two halves of a rectangular field of view. For color-matching, it is merely necessary to move the slide *A* in its groove until the two halves of the field of view have the same color. The color number is read off the scale *S* directly.

A rather remarkable range is possessed by this instrument. The amber wedge is 17 cm. long, 0.1 mm. and 4 mm. thick at opposite ends, while the equally long hollow wedge is 0.1 mm. and 10 mm. thick at the ends. Since the wedges are *reversed*, their combined effect is equivalent to a single wedge of amber glass whose thickness ratio at the two ends is 4000:1.

As for the character of the color scale, it may be said that, if the colored medium was black and hence ranged from transparency through the grays to opacity, a rational logarithmic color scale, based on Beer's law, could be constructed. In other words, the color number of two samples would be in the ratio of, say, 2:1 if the percentages of coloring matter were also in the ratio of 2:1. In the case of varnishes, however, increasing amounts of coloring matter increase not only the opacity and the "purity" of the transmitted light, but affect also the "dominant wave-length." It is conceivable that this complication might lead to a disparity between a color-scale based on Beer's law and a subjective color-scale such as that of Gardner and Holdt.[1] For the time being, the latter scale will be adhered to. The instrument is thus calibrated by filling the hollow-wedge with the several Gardner-Holdt caramel solutions and finding the respective positions of color-match on the scale. A check on the reliability of the instrument is possible at any time by filling the hollow wedge with a solution of 3 g. of potassium dichromate in 100 cc. of sulfuric acid (sp. gr. 1.84). If the instrument reads 9.0 at color-match, no change in the amber glass wedge has taken place.

Concerning the accuracy of setting, it may be said that the color number of any varnish may be determined to within a few tenths of

[1] Gardner and Holdt, "Physical and Chemical Examination of Paints and Varnishes," 2d Ed., p. 164.

a unit. As a result of using a disk of diffusing glass for purposes of illumination, the observer is not confused by the presence of suspended matter in a varnish sample. So long as the sample in the hollow wedge is not illuminated on the side nearest the observer, the resultant color transmitted is not affected by the presence of suspended particles. The latter will, it is true, decrease the total amount of transmitted light but will not affect the "yellowness."

LEVELING OF VARNISHES

After spreading varnish over a surface, a brush may be passed over the surface for some time without leaving permanently visible

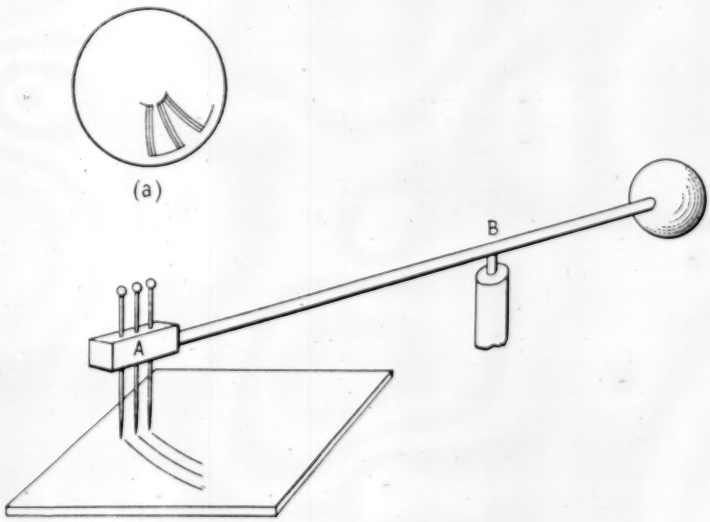


FIG. 5.—Apparatus for Determining Leveling Properties of Varnish.

brush marks. Failure to obliterate these marks sets in as soon as the "yield value" becomes greater than the surface tension. In order to measure this "leveling" property of varnishes, instruments have been devised for drawing slowly a freshly varnished plate under a stationary metallic comb whose teeth pass through the varnish layer. Because of the slowness of the motion, the varnish which piles up in front of each tooth loses so much of its volatile constituents before flowing around to the other side of the tooth that failure to level sets in prematurely. It was sought, therefore, to modify the procedure so as to simulate more closely the rapid motion of a brush.

In Fig. 5, the principal features of the apparatus are shown. Here, *A* is a brass block bearing three vertical holes 2 mm. apart

which serve as guides for stout needles. Contact of the needle points with the varnish-covered glass plate is assured through the use of small lead weights resting on the upper ends of the needles. This system is attached to a rod pivoted to a vertical axis *B*. Since varnish films of uniform thickness are so readily obtained by the whirling disk method, it was deemed desirable to adapt the present instrument to the use of circular plates. Consequently, the arm *B* is swung rapidly across half of the plate, that is, from center to edge. The plate is then rotated through 30 deg., and after five minutes, the motion is reversed. The type of record left by the needle-points is shown in the insert Fig. 5 (*a*). Failure to level is revealed—not by the deep, permanent furrows left by the needles—but, by the faint undulations which are detected by holding the plate off at arms length and reflecting the light from a window into the observer's eye at nearly normal incidence. It may be stated that the instrument brings out very definitely the different leveling properties of various varnishes.

Thusfar the operation has been carried out by hand. It is planned, however, to make the process automatic so that the time interval elapsing between the application of the varnish and the first failure to level may be read off the glass disk directly.

In conclusion it may be stated that these instruments are all of such recent origin that an insufficient length of time has elapsed to reveal inherent weaknesses or possible fatal defects. The entire program is therefore to be looked upon as a tentative suggestion only, for, as has been stated previously, it is felt that the complete solution of the problems in hand can be arrived at only as a result of cooperative effort.

The preceding work was carried out, largely, at the Experimental Station of the E. I. duPont de Nemours Co.

DISCUSSION

Mr. H. A. Nelson.[1]—There is one point about the hardness test described by Mr. Pfund that it seems to me should be made clearer. Mr. Pfund has undoubtedly considered the effect of film thickness on hardness measured acording to his sphere method, but it is not discussed in the paper. **Mr. Nelson.**

In the formula $D = \sqrt{4\rho t}$, given in note 2, bottom of page 395, it will be seen that the thickness, t, is the only variable, and that the results are dependent on the accuracy with which this thickness can be reproduced. Could you tell us how closely this thickness can be regulated for different varnishes?

Mr. A. H. Pfund (*by letter*).—Replying to Mr. H. A. Nelson, in making measurements with the hardness tester, it is tacitly assumed that the steel ball does not penetrate the varnish film completely until contact is made with the underlying plate of glass. The formula referred to by Mr. Nelson is given merely as a guide to prevent the above-mentioned penetration. The measurements on film-thickness do not, therefore, enter into the measurements of hardness. **Mr. Pfund.**

Mr. Nelson.—This clears the question that came to my mind when I read the preprint. **Mr. Nelson.**

Further, concerning the distinction between hardness and toughness of paint, varnish and lacquer films, this is an interesting problem about which we do not have as much information as we need. The length and relative position of the stress-strain curve certainly tell us much about what we think of as the toughness of the film, although we do not yet know if they tell the whole story from a practical standpoint. Rubber technologists, however, generally accept the energy area under the stress-strain curve as a measure of relative toughness of rubber, and, considering that rubber and drying oil films have so many physical properties in common, we undoubtedly are justified in making a similar assumption for paint and varnish. Probably this will apply to lacquer films as well, but the physical properties of these films should, I believe, be studied further before stating conclusions.

[1] Investigator, Paint Section, Research Division, New Jersey Zinc Co., Palmerton, Pa.

Mr. Little. MR. W. F. LITTLE.[1]—I should like to ask the following questions in reference to Mr. Pfund's paper:

1. What method is there of correlating the results when different angles of incident light are employed? For instance, when in one sample the angle of 45 deg. is used, and in the next sample it is necessary to use a much larger angle, what is the relation of the results of one test to that of the other?

2. What advantage has this method over the polarized light method which is in general use to-day, and how can the two sets of results be compared?

3. Is not the best method that of measurement of light at, say, 45 deg. and at 90 deg. removed? In that way the approximate relation of specular to total is secured.

4. Is not gloss the relation of total less diffuse to total and will this instrument give that result? It would seem advisable to get some method that all laboratories may use and which will give comparable results under all conditions.

5. The method described in the paper if applied to metals, such as silver, nickel and aluminum (the specular reflection component is high in all three cases, but the overall reflection factor differs widely), would give substantially the same result for all three, whereas their gloss will differ considerably.

6. In Fig. 2 of the paper is shown a small window, 3 mm. in diameter in the illuminator; suppose the sample is not absolutely flat, is it possible to secure accurate results? If that aperture were 3 cm. instead of 3 mm., there would be no question of receiving all the light in the cube. In this case a slight variation in the surface would certainly throw out the uniformity of the field.

7. With reference to color, the paper refers to the use of a glass wedge against which is compared the color of the varnish. Is it possible to get one glass which will take in the complete range of color?

8. Do not the varnishes range from yellow to red and is not the matching by the method shown in Fig. 3 more nearly transmission than color? By sliding the slots across the wedges a photometric balance is secured and not a color match.

Mr. Pfund. MR. PFUND (*by letter*).—Replying categorically to the questions of Mr. Little.

1. While the possibility of increasing the range of the gloss instrument by increasing the angle of incidence is suggested, it is found in practice that the 45-deg. angle of incidence covers practically the entire range from high-gloss to flats. Therefore, while no correlation

[1] Electrical Testing Laboratories, New York City.

of gloss readings at different angles of incidence has been worked out, there is no reason why it can not be done. **Mr. Pfund.**

2. Replying to the second query,—a most important question,—the present instrument was developed only after exhaustive experiments with the polarized light instrument had been carried out. It developed that the latter instrument gave identical readings for black enamels differing very widely in gloss. For the purpose of making fine distinctions between surfaces of very high gloss, the polarized light instrument is useless.

3. It must be remembered that we are dealing, most frequently, with a black background over which a layer of clear varnish has been flowed. Measurements carried out at 45 deg. for diffuse reflections would reveal that this intensity is, to all intents and purposes, zero.

4. Insisting once more that the present instrument is for varnishes only, the answer to this question is "yes"—if the "normal" gloss scale is considered. As for the high-gloss scale, it was felt that, since no existing instrument was capable of making the desired fine distinctions, it was worth while to attempt to find a criterion which would make the measurement of these small differences possible. The circumstance that this new "high-gloss scale" bore no necessary relation to the accepted gloss scale seemed to us a matter of minor importance.

5. The present instrument was designed for varnishes only—not for metals. By carrying out measurements on specular and diffuse reflection, the instrument could, however, be used to study the gloss of metals without making any changes in the construction.

6. The question of planarity of the surface to be tested is of great importance. Realizing that plane surfaces of varnish could be produced readily by flowing the varnish on plane sheets of glass or metal, the present test was devised. Needless to say, it will not work if the surfaces are distorted or corrugated.

7. The answer to the seventh question is "yes."

8. In making readings on the varnish color apparatus, one forgets all about intensity differences. It is merely necessary to find positions of the amber wedge in which it appears, first, "paler" than the varnish and then, "yellower." It is surprising to note how accurately the settings can be made even by untrained observers.

MR. F. C. SCHMUTZ.[1]—In regard to the question whether polarized light could be used in this instrument, I wish to point out that measurements of the same gloss taken over different colored backgrounds are not comparable when read with the aid of polarized **Mr. Schmutz.**

[1] Investigator, Paint Section, Research Division, New Jersey Zinc Co., Palmerton, Pa.

Mr. Schmutz. light as is done in the Ingersoll glarimeter. From our own observations these readings seem to have a geometrical relation to one another; they probably could be made comparable by means of a correction factor. I do not believe any such correction factor has been developed as yet.

A RECORDING DRYING-TIME METER FOR VARNISHES AND SIMILAR MATERIALS

By J. McE. Sanderson[1]

Synopsis

In an effort to eliminate the personal equation in testing the drying time of varnishes and similar materials, the author has developed mechanical means for measuring and recording this quality. In doing so, he has utilized the method of Walker and Thompson to secure uniform films of the materials to be tested. The mechanism for recording changes in the drying films is adapted to a wide range of materials, from those which dry in a few minutes to those which require up to ten days, and it is of sufficient simplicity to be available for ordinary routine testing. The recording mechanism eliminates many of the faults of the ordinary methods of testing now in general use, and paves the way for further study of the many variable conditions which affect the drying of films of varnish and similar materials.

Many manufacturing processes depend for economical operation on quantity production at a scheduled speed. Each part of the process must keep step with the rest. The application of paint or varnish for decoration or protection is an important part of the manufacture of many articles. It generally comes near the end of the process where a disarrangement of the schedule would cause the most confusion. Hence the commercial value of a surface coating is often determined by the rate at which it dries. It has been shown[2] that drying tests on the same materials by different operators do not agree closely and that the various stages in the drying period are not very exactly defined. Hence the importance not only of more exact means of measuring drying time but also of some means of recording these measurements.

Up to the present time, no satisfactory method has been devised for absolute standardization of all the variables such as heat, light, humidity, etc., which affect the drying of surface coatings. This subject is to be studied by the Society's Committee D-1 on Preservative Coatings during the coming year. No attempt has, therefore, been made by the author to record in this paper the effect of these

[1] Manager, Paint Division, Larkin Co., Inc., Buffalo, N. Y.

[2] Report of Sub-Committee IX on Varnish of Committee D-1 on Preservative Coatings for Structural Materials, *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part I, p. 375 (1922).

variables, all tests having been made under the ordinary indoor drying conditions prevailing at the time the tests were made. Drying tests are generally comparative rather than absolute. The usual method is to apply the material under test together with a standard material on similar surfaces and to touch it at intervals. The coating is considered "dry to touch" when a gentle pressure shows it slightly tacky but none adheres to the finger. It is considered "dry hard" when the pressure which can be exerted between the thumb and finger does not move the film or leave a mark. In the drying test as ordinarily carried out, the personal equation has a large influence on the results obtained: first, because of the difficulty of securing uniform application of two or more different materials, and, second, in judging when a film has reached any definite stage of drying. Constant attention is required to catch the right end points, and under ordinary working conditions, it is difficult to check the drying of material requiring more than the daytime interval of 8 hours and less than the overnight period of 16 hours.

Several attempts have been made in the past to devise mechanical means for testing and recording drying times. The best of these is undoubtedly the one devised by H. A. Gardner[1] which had, however, enough defects to prevent its practical application. Using this machine as a basis from which to work, the author has attempted to:

1. Provide mechanical means for securing uniform application;
2. Record mechanically changes in the material as it dries;
3. Adapt this mechanism to a wide range of materials from those drying in a few minutes to those requiring a week or more; and
4. Make this mechanism of sufficient simplicity so that it would be available for ordinary routine testing and not be merely a laboratory curiosity or useful only for elaborate research work.

Preliminary investigation failed to disclose any method of application superior to that of Walker and Thompson[2] whereby the material is spread by centrifugal force when poured on near the center of a rapidly rotating disk. This method is applicable to varnishes and oils and to most enamels and paints. Viscosity and other qualities appear to have at least no greater influence than on the film secured by brushing, dipping, or spraying under practical working conditions with the same material. This method not only applies the material mechanically in a uniform manner, but it also applies it to a plane surface of sufficient size to give a reasonable idea as to the covering capacity. The disks may be of a variety of materials and they may

[1] *Bulletin No. 167*, Paint Manufacturers' Assoc. of the U. S., Educational Bureau.

[2] P. H. Walker and J. G. Thompson, "Some Physical Properties of Paint," *Proceedings*, Am. Soc. Testing Mats., Vol. 22, Part II, p. 464 (1922).

be subjected to any of the usual range of drying conditions, even to baking if desired.

Starting with the coated disks, the next step was to devise a mechanical means of continuously touching the film as it dried. Bringing it into contact with a powdered solid, such as sand, seemed to eliminate at once the variations in pressure and the necessity for an absolutely plane surface free from specks which influence so largely tests by contact with strips of paper or blocks of metal. Gardner had tried dropping sand intermittently, but experienced mechanical difficulties in doing so. The writer eliminated this trouble by lowering the funnel containing the sand so that the tip was within about $\frac{1}{16}$ in. of the disk. In this position, the sand would run only as the disk moved under the tip of the funnel.

With this as a basis, the first crude machine was made up. The disks were of tin and were coated while revolving at 500 r.p.m. On the testing machine, the disk was driven one revolution in twelve hours by an alarm clock movement. The sand was contained in a glass funnel, hung so that the height above the disk could be regulated by thumb nut (although, as it swung in an arc, the height above the disk varied). As the disk rotated, the tip of the funnel was pulled toward the center by a thread winding on the clock spindle so that the sand was deposited in the form of a spiral. This machine showed promising results. The sand left on a varnish film after a 48-hour test when the disk was turned upside down and tapped vigorously, seemed to indicate time of drying "free from tack." The sand remaining on the same disk after brushing seemed to indicate the time of initial set. The writer discussed these results in a very informal way at the meeting of Committee D-1 in June, 1924, and was encouraged by the comments at that time to proceed further with the work.

The original machine was found to have a number of serious defects. It made no provision for the camparative testing of two or more materials. The alarm clock mechanism used for drive would not run for more than 48 hours. The disk was not sufficiently well supported, and the flow of sand could not be closely regulated. Most of the work during the past year has been devoted to overcoming these mechanical difficulties and designing a machine which would do the work intended and still be of sufficient simplicity and low cost to be available for ordinary routine testing. After some difficulty, a clock movement of sufficient strength to drive a more complicated mechanism was secured. After trial of a number of other designs, a machine was constructed in which three disks were carried on steel phonograph tables rotated by the single clock at a speed of one

revolution in twelve hours. The sand funnels were suspended from a rod moved horizontally by a gear-driven screw. Moving the funnels in a straight line instead of swinging in an arc, as in the first machine, allowed a closer regulation of the flow of sand. Its construction required numerous bevel gears, ball bearings, and very accurate alignment, and was consequently rather complicated and expensive.

In the next development most of the expensive construction of the previous machine was eliminated by driving each disk with a

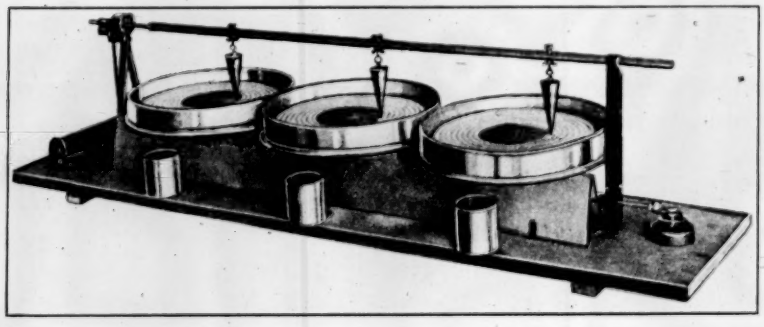


FIG. 1.—Combined Coating and Testing Machine, Complete.

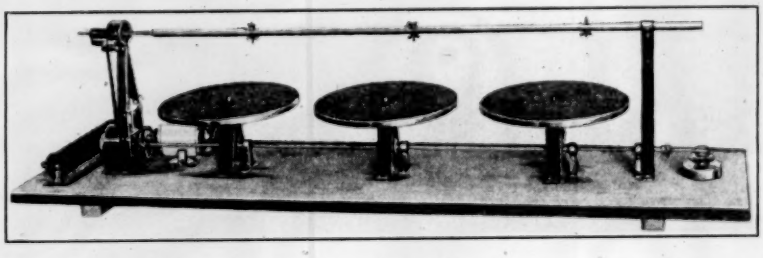


FIG. 2.—Combined Machine; Cover Removed to Show Construction.

separate clock and moving the funnels with a string. We were limited however, on this machine to a single speed of the disks, which in this case was one revolution in 24 hours. As the rotating tables were attached directly to the clocks, it was difficult to adjust them to rotate accurately in a horizontal plane. The effect of poor adjustment is seen in Figs. 4 and 5.

Tests on the last two machines indicated that the time required for transfer of the disks from the motor-driven coating mechanism to the clock-driven testing machine, was an appreciable part of the drying period, especially of such materials as rubbing varnish, shellac.

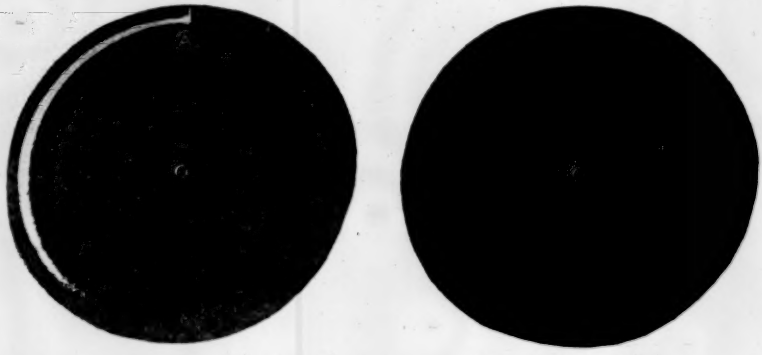
etc.; also that to cover the wide range in materials from spirit varnishes to oils, would require a widely varying range of speed of the disks. A speed of one revolution in 6 hours or less seemed to be desirable to distinguish between quick-drying materials and a much slower speed, not over one revolution in 24 hours, for oils and other materials requiring a week or more to dry.

To overcome these objections, the machine shown in Fig. 1 was built, in which was combined the coating and testing mechanisms. Fig. 2 is another view of the same machine with the cover removed. In this machine, the disks are revolved for coating by an electric motor (taken from a dictograph) which can be regulated very closely to any speed between 200 and 500 r.p.m. The speed which we used on most of our tests was 350 r.p.m. By a simple shifting of gears, the drive can be changed from the motor to clockwork which rotates the disks once in 12 hours. After this picture was taken, another gear was added to the clock to give a second adjustment of speed to one revolution in 6 hours. With this machine, three disks can be coated simultaneously, the time required ranging from 15 seconds to two minutes, depending on the nature of the material. After coating, the test can be started within less than one minute.

Even a disk speed of one revolution in 6 hours seemed to be too slow for testing quick-drying varnishes and lacquers. To test these, the original machine was reconstructed to run the disk at the rate of one revolution per hour by attaching it to the minute hand spindle. At this speed with a disk $12\frac{1}{4}$ in. in diameter, the first line of sand is deposited at the rate of about $\frac{5}{8}$ in. per minute, which allows of fairly sharp distinctions among quick-drying materials. A speed of one revolution in 6 hours appears about right for short oil varnishes, once in 12 hours for long oil varnishes, and once in 24 hours for very slow varnishes and for oils. About $\frac{1}{2}$-in. movement toward the center must be allowed for each turn of the disk which allows about ten turns on a $12\frac{1}{4}$-in. disk.

Having developed in the machines described above the features which appeared to be essential for a satisfactory mechanism, a further endeavor was made to simplify and cheapen the design while still retaining all the desirable features, including flexibility. In this design the disks were driven with chain and sprockets by an electric motor at 300 r.p.m. or by clocks at speeds of one revolution in 1 hour, 4 hours, 12 hours and 24 hours. To do this requires the use of two clocks with two different gears on each. Pans are provided (similar to those on the machine shown in Fig. 2) to catch the excess varnish thrown off in coating the disks.

Various grades of sand have been tried, as well as emery, but no especial difference in the way they adhere to drying films was found. But for the sake of getting a dependable, uniform flow through a small orifice, we have found it desirable to use round-grained sand mixed with a little emery to darken the color, sifted through a 60-mesh and retained on an 80-mesh screen. To maintain a steady flow over a period of several days, it is necessary to have both the sand and the funnels thoroughly clean and dry. The disks used for most of the tests have been either tin or double-strength window glass. Ordinary glass is satisfactory if care is taken to select pieces that are flat. Plate glass is rather heavy and expensive for ordinary use. Tests can also be run on fiber board, wood, or other materials on which



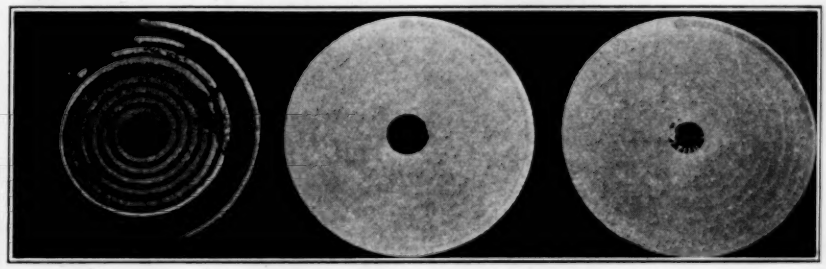
(*a*) Before Brushing. (*b*) After Brushing.

FIG. 3.—Gloss Oil. Disk Run One Revolution per Hour.

the coating is to be applied in actual use. Disks of material which tends to warp must be kept flat by clamping to the rotating tables. It is possible to apply the test after two or more coats of finish have been applied. These features are important, as finishes sometimes dry differently on different surfaces and the second coat differently from the first.

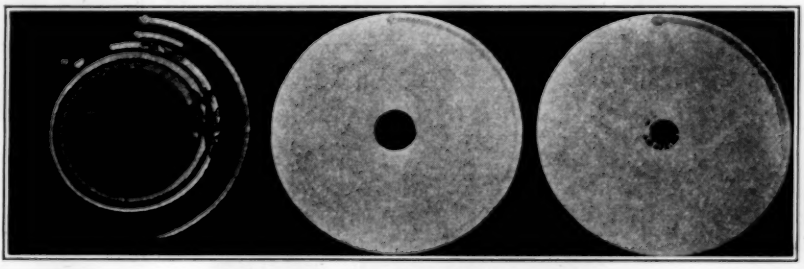
Tests by the method outlined above seem to determine a fairly definite degree of drying which we call "dry free from tack." This point is indicated by the sand which adheres to the surface after the disk has been turned upside down and tapped sharply to remove the excess. The point which we would call "dry to touch" is then determined by brushing the disks vigorously with a bristle paint brush which removes the sand particles that are not actually embedded in the varnish film.

Although we have made numerous tests in determining the proper design of the machine, we do not have tests on a sufficient variety of materials by different operators to feel that we can draw definite conclusions. We expect to place these testing machines in the hands of a number of members of Sub-Committee IX on Varnish of Committee D-1 for work along this line during the coming year. The



(*a*) Raw Linseed Oil. (*b*) Long Oil White Enamel. (*c*) White House Paint.

FIG. 4.—Disk, Run One Revolution in 24 Hours; Before Brushing.

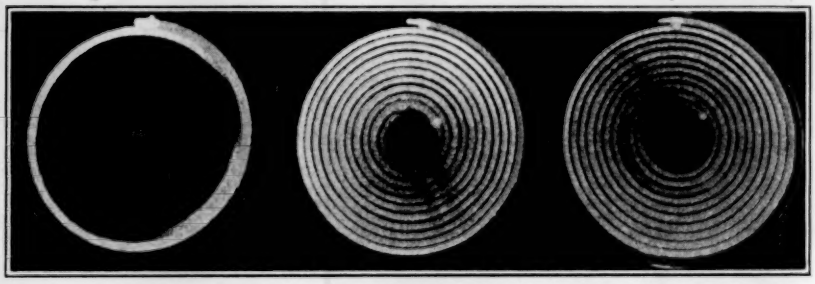


(*a*) Raw Linseed Oil. (*b*) Long Oil White Enamel. (*c*) White House Paint.

FIG. 5.—Same as Fig. 4 after Brushing.

results so far obtained point to very interesting psosiblities for study. The author feels that much can be learned by the study of the way in which the sand adheres both before and after the films are brushed. Films of spirit varnish and flat paints which dry mostly by evaporation of the volatile thinner show a fairly sharp end point beyond which the sand does not adhere. Fig. 3 shows results on gloss oil with the disk running one revolution per hour, (*a*) before brushing and (*b*) after brushing. On the other hand, house paints, oils, and outdoor varnishes which dry largely by oxidation show a considerable period during

which the adherence of the sand gradually diminishes and a considerable difference before and after brushing. Fig. 4 shows a nine-day test with disks run one revolution in 24 hours: (*a*) raw linseed oil, (*b*) long oil white enamel and (*c*) outside white house paint. Note adherence of sand for full time of test although disks have been turned over and tapped to remove excess. Fig. 5 shows same disks after brushing. The effect of faulty adjustment is clearly shown by disk (*a*). In this



(*a*) Removed after 72 hours. (*b*) Removed after 8 days. (*c*) Removed after 8 days.

FIG. 6.—Check Tests on Same Varnish before Brushing. Disks Run One Revolution in 12 Hours.

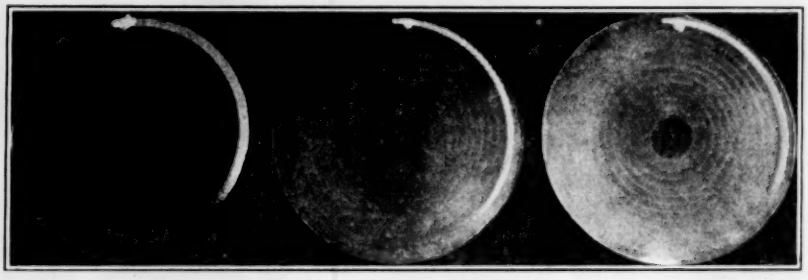


FIG. 7.—Same Test as in Fig. 6 after Brushing.

case the rotating table did not run true, allowing the disk to touch the tip of the funnel and clog the sand with a drop of oil. It had to be cleaned out and started twice before it ran smoothly again.

Tests on the same material run on several disks simultaneously have checked very closely both before and after brushing the disks. On the other hand, a slight variation of conditions makes a considerable difference in the way the sand adheres before the disks are brushed. Fig. 6 shows three disks on which the same varnish was tested simultaneously with the disks rotating once in 12 hours. One disk was removed after 72 hours, turned over and tapped to remove excess sand.

As noted in the cut, the sand adhered only for 18 hours. The other disks were allowed to run for five days and the sand to stand on them for three days longer. When they were then turned over and tapped, the sand adhered for the whole five-day run. On the other hand, after a vigorous brushing, all three disks showed close agreement in time of initial set as shown by Fig. 7. This indicates that, working under exactly identical conditions, the adhesion of the sand before the disks are brushed will probably give some information of value, but the main conclusions as to drying are more likely to be based on appearance after the loosely adhering sand is brushed off. On the other hand, in a comparative test of two or more materials any radical difference in the retention of a slight tack or stickiness over a long period will be revealed. This is often a source of trouble which it is well to be able to detect.

THE DESIGN OF A RESEARCH LABORATORY FOR A TEXTILE MANUFACTURING PLANT

By G. B. Haven[1]

The recent rapid industrial development in this country has been in large measure the result of the employment of scientific agencies for originating new methods as well as improving existing processes, whereas, heretofore many manufacturing methods had resulted simply from years of experience and practical experiment. The employment of scientific methods as an aid in the industrial arts results not only in the cooperation of technically trained men with experienced manufacturing executives but also in the development of industrial laboratories maintained in the factory or mill itself. Every industrial operation presents questions of a scientific nature. Results of value can rarely be obtained aside from extended experiments and practical research. When such experiments are carried out by experts outside of the plant the expense involved is frequently very great. If such experiments are carried out inside of the plant, but without definite aim and correct scientific methods, the results are often misleading. But with a laboratory maintained in the factory, the work of the laboratory is brought into as close relationship as possible with the conditions necessitating the scientific investigation.

It has long been considered that research laboratories to be of any value necessitate a very considerable expenditure of money and require the supervision of a highly paid scientific staff. While in a measure the personnel of such laboratories must be of a high grade of intelligence, it is at the same time possible for one or two skilled research men, with a small clerical and executive staff under their direction, to carry out very extensive programs of research and render a continuous and far-reaching service to every class of industry. With the latter object in view, the author has been called upon at numerous times to design and superintend the equipment of industrial laboratories for textile and other manufacturing plants.

When the size of the industry would limit the entire scientific staff to one well-trained man, he should of necessity be a chemist with as extensive a knowledge as possible of mechanical engineering. With two technical assistants, one a chemist and the other a mechan-

[1] Professor of Machine Design, In Charge of Textile Laboratory, Massachusetts Institute of Technology.

ical engineer, and a stenographer for general clerical work, research laboratories may be maintained at a minimum of expense but with very considerable effectiveness.

When the size of the industry warrants the outlay, it is always wise to subdivide research questions into two groups, namely, physical and chemical. The physical questions usually deal with the purely mechanical properties of the manufactured product, such as strength, elasticity, uniformity of weight and form, and the general ability of the product to sustain the wear and tear for which it was designed. The chemical questions involve more profound research in the direction of atomic composition, deterioration by time and use, and frequently biological questions regarding the effects of mildew, decay or accidental chemical action.

It has been found rather difficult to train one man to have the insight and attributes of a pure chemist, while possessing at the same time the intensely practical mind of an engineer. Hence, the above subdivision is very desirable in the case of an industry of considerable importance. It is needless to say that these two departments should work in entire harmony with one another while pursuing each its own line of research. One technical assistant for each department, together with a stenographer to be used in common, will be a sufficient staff to carry out very extended programs of experiment and research.

It should be remembered that the most successful industrial research laboratories to-day are those which are equipped not alone for the study of all routine questions arising in a given line of manufacture, but are also able to handle extraordinary subjects which may arise. The objects of industrial research are two-fold. Primarily the manufactured product is produced of better and more uniform grade and at less expense when safeguarded by thorough inspection and test. Secondly, a much more uniform and satisfactory grade of raw materials such as coal, oil, water, fiber, chemicals and general supplies for the plant may be secured when it is known in advance that these commodities will be subjected to a rigid test and refused if undesirable. A research laboratory should, therefore, include in its scope the testing and standardization of every known and expected material which goes into the make-up of a given manufactured article.

In the physical laboratory, equipment should be provided for complete tests upon the mechanical properties of all materials and manufactured products. The chemical laboratory should handle all questions of analysis, reaction, and chemical properties and relations.

As a provision for future extension, a well-equipped laboratory should provide lighting and industrial alternating and direct electric

currents of various voltages, compressed air at any adjustable uniform pressure desired, hot and cold tap water of fair purity, distilled water for refined operations and illuminating gas for heating as well as lighting. Added to this should be portable oxygen tank service for experiments in combustion.

TEXTILE LABORATORY

Taking the textile laboratory first in order, the equipment should include the following apparatus.

1. General hot water system of heating for the laboratories, the same to be under thermostatic control at a variable and adjustable level of room temperature.

2. Air conditioning apparatus to raise and hold automatically the atmospheric humidity to a fixed standard.

3. Dehumidifying apparatus to reduce the natural atmospheric humidity when necessary to a definite standard.

4. A continuous psychrostat permanently mounted at the level of the conditioning rack. This instrument should be arranged with a small fan and wet and dry bulb thermometers in such manner that it may be run for long periods and will indicate continuously the relative humidity of the atmosphere.

5. An automatic electric drying oven for use in determining the moisture content of fibers, fabrics and other materials.

6. A vertical autographic yarn testing machine with dials calibrated from 3 to 50 lb. for testing single strands of yarn, the autographic recorder being used to make a permanent record of the stretch through the various loads to the rupture.

7. A vertical fabric testing machine with autographic recorder having calibrated dials from 50 to 300 lb. capacity. This machine is adapted to testing yarns in skeins or leas of any reasonable yardage.

8. A heavy horizontal testing machine for strong fabrics and other materials, with autographic recorder, calibrated to 800 lb. capacity.

9. A circulating fan whereby a moderate but uniform current of air may be blown throughout the laboratory in such a manner as to prevent the stratification of moisture as far as possible.

10. A conditioning rack on which textile specimens to the number of several hundred may be hung when bringing them to condition. This rack should slowly rotate, being driven by an independent motor in such a manner as to bring both sides of the samples under the full effect of the atmospheric moisture in the room and cause the fabric to come promptly to condition.

11. A cutting table made of blocks of hard pine with grain on end in such manner that a razor blade may be used to cut weight samples and test pieces accurately without sailing in the grain of the wood.

12. An illuminated inspection table. This piece of apparatus consists of a sheet of ground glass 2 ft. or more square with numerous electric lights below so placed as to illuminate the ground glass uniformly on the underside. This "light table" is very useful in counting threads, taking magnified photographs of fabric and inspecting goods for imperfections.

13. A two-cylinder air pump and air tank so arranged that the air pump will be started automatically at a determined limit of pressure in the tank, this limit being adjustable and easily set to any desired amount. This rig is necessary in connection with the humidity control and is very desirable in order to furnish a steady and dependable volume of compressed air for experimental purposes.

14. A drafting table provided with parallel ruler and individual light, the top being large enough for use in laying out extensive plots and other experimental data.

15. Small desk for laboratory director with telephone connection with heads of all manufacturing departments.

16. Electric motor of not less than 5 h.p. with line shaft on the wall for driving testing machines and other apparatus. It is very desirable that this be a direct current motor with a speed-control box in circuit so that the testing machine speed may be varied within wide limits.

17. Balance Room. Fine balances and a binocular microscope should be provided, preferably in a small room accessible from both the physical and chemical laboratories, but shut off from both. This is necessary because of the injury to such fine apparatus by the excessive moisture in the textile laboratory. A deep concrete foundation entirely independent from the rest of the structure should be provided for the balance and microscope table. The balance should be of the "chainomatic" type with a full complement of gram and grain weights, the rider being calibrated for both. The balance room should contain abundant shelf room for desiccators, microscope slide cases and the materials necessary in making microscope slides. The fine balances should be illuminated with a hooded light permanently fixed to give the best possible illumination of the rider scale.

18. Six sections of bookcase for a textile library, with four-drawer letter file for pamphlets.

19. One bursting tester of 300 lb. capacity for determining the strength of paper or fabric under bursting strain.

20. One yarn-examining machine with 12 blackboards. This is useful for laying side by side numerous windings of yarn in order to judge of relative unevenness, knots, snarls, etc.

21. Window fan electrically connected, with slide so as to admit outside air into room when desired.

22. Slate-stone sink with hot and cold water.

23. Control board with gages and adjustments for starting and stopping air compressor and controlling the pressure automatically.

24. Instrument case with the following equipment of small apparatus for general utility in testing fabrics and yarns:

(*a*) Pick-glass for counting threads per inch.

(*b*) Yarn reel, rotary type, for skeining from bobbins, cones, etc.

(*c*) Yarn reel, umbrella type, for reskeining and measuring loose yarns.

(*d*) Twist counter with magnifying glass for determining twists per inch and plys in compound yarns.

(*e*) Roving reel for measuring length of roving and coarsely twisted yarns and cords.

(*f*) Sliver board for measuring sliver weights and very coarse roving.

(*g*) Photo-trimmer for cutting samples of paper, etc.

(*h*) Weighing scales consisting of balances accompanied by a series of weights up to 5 lb. by quarter ounces, and a series of gram weights up to one kilogram. This scale should contain two riders, one for ounces and one for grams.

(*i*) One electric flat iron for pressing samples.

CHEMICAL LABORATORY

The chemical laboratory should be subdivided into the following general departments:

(*A*) General Chemistry.
(*B*) Fuel Analysis.
(*C*) Photography and Lantern Slides.

(*A*) GENERAL CHEMISTRY

The laboratory of general chemistry should include the following apparatus:

1. Chemical filing case for not less than 450 bottles of chemicals, each with separate compartment and label. These cabinets are stock and may be secured with unit locks throughout in fine finish of hard wood.

2. Water still. This apparatus should be equipped with large gas burner, automatic water feed, and cooling coil for producing a general supply of distilled water for both laboratories. If gas is not available this still will need to be run by an electric heater. It would seem advisable in this event to install a small independent gas machine, since the latter would be a necessity in chemical work.

3. Long lead-topped chemical table with 4-ft. slate-stone sink in center. This bench should be provided with universal electric, compressed air, gas, steam, tap and distilled water service. Drawers and cupboards should be installed underneath.

4. Flat top desk for superintendent.

5. Eight-foot ventilated hood with small slate-stone sink and lead-topped table.

6. Six sections of bookcase for chemical library, with four-drawer letter file for pamphlets.

7. Instrument case for holding chemical apparatus, general supply of glass and rubber tubing, test tubes, Bunsen burners, beakers, lamp stands, small centrifugal machine, filter paper and other general chemical apparatus and supplies.

8. High shelf for carboys of standard solutions with pippette service.

9. One-fourth-horsepower electric motor and small line shaft over chemical benches for driving stirrers, centrifuges, etc.

(*B*) COAL ROOM

1. Five-foot quartering table for selecting samples of coal.

2. One coal grinder for pulverizing fuel samples, belt driven from 1-h-p. motor on wall.

3. One coal calorimeter or bomb for determining heat of combustion.

4. One portable oxygen tank for B. t. u. determinations.

5. One small exhaust fan in window for removing dust.

(C) PHOTOGRAPHIC ROOM

1. A dark room should be supplied with large built-in red lantern and red window panes, trays, sink, etc., for developing photographs. A dark ventilator with electric fan should be provided. Shelves for chemicals, and a foot treadle for making exposures in lantern slides.

GENERAL DESIGN

A hip-roof building facing the south, constructed of either wood or brick with a low studded basement is very desirable for the purpose of these laboratories. At the entrance on the south side should be a general waiting room with typewriter desk and local telephone exchange. On either side of this waiting room should be the offices respectively for the directors of the textile and the chemical laboratories. It is generally considered best to place this research work under the direction of two men, one for the textile and the other for the chemical side of the question. The director of the textile side should have general charge of all mechanical questions and difficulties arising in the plant and should be capable of testing the physical properties of all the raw materials and manufactured products of the industry. The director of the chemical laboratory should be responsible for the chemical analysis of raw materials and manufactured products, all questions of sanitation, health and welfare among the employees and all questions arising from physical injury, hospital maintenance and the purity of the food and water. The two directors of this laboratory should work in thorough cooperation and should aid one another in every way by their respective services.

The laboratories should be on the north side of the building, with few windows if any in a southerly direction, but with very large windows on the north side, the sills of which should be at least 6 ft. from the floor. The glass in the upper third of the windows should be factory ribbed. The chemical side should occupy the northwest side of the building, since the heat of a western sun is of less consequence in chemical work than in textile work. Under the windows in the north wall there would be plenty of room with fine illumination for testing machines and chemical benches. All the windows in the textile laboratory should be double sashed with a dead air space between. This would insure stationary conditions as far as possible for testing textiles. The air compressor and tank and gas machine, if one is used, should be accomodated in the basement and there should be abundant room there also for storage, general conditioning and aging operations, lavatory and toilet room.

In each of the directors' rooms is provided a desk for a technical assistant. The staff would therefore include four persons of scientific

training and a fifth as general stenographer, clerk and telephone operator. In the headquarters of the textile department is provided

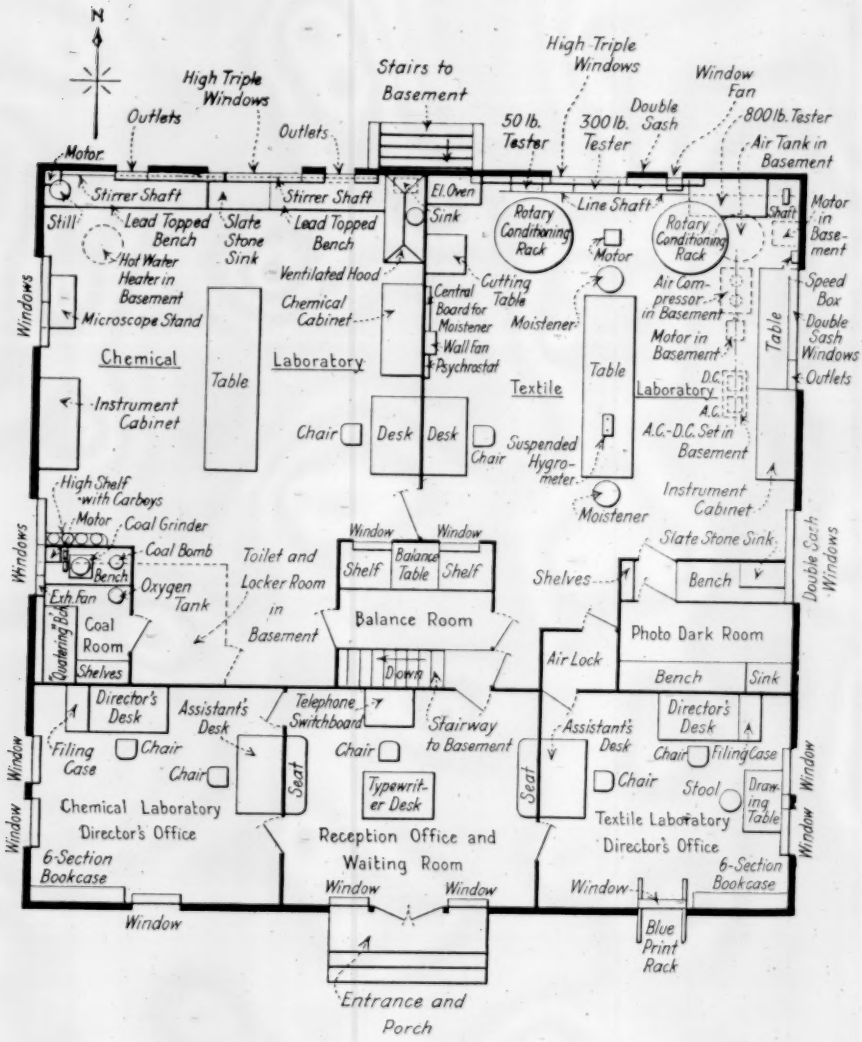


FIG. 1.—Design of Research Laboratory for Textile Plant.

a drawing table and blue print frame so that plots, charts, etc., may be printed directly from sun light in writing reports, etc.

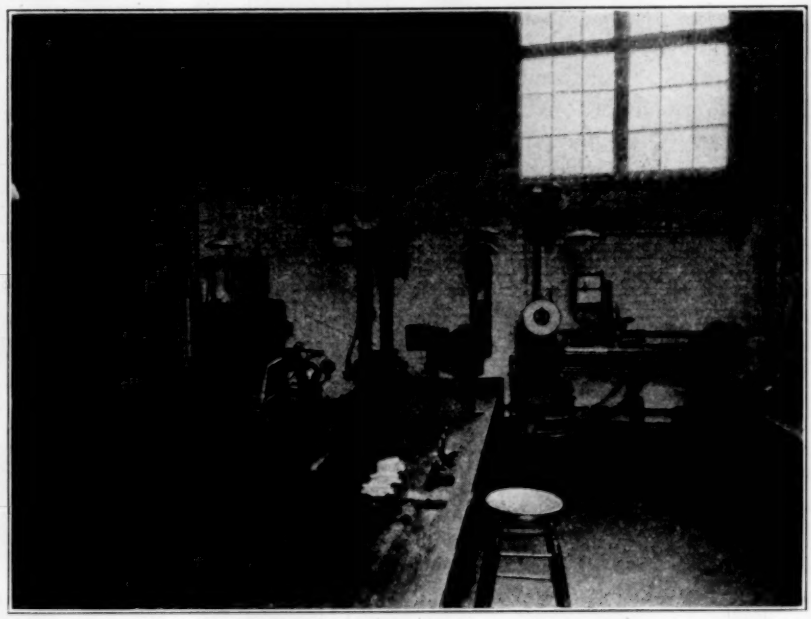


FIG. 2.—Interior of Textile Laboratory.

FIG. 3.—Interior of Chemical Laboratory.

In view of the above general statement of requirements the general plan shown in Fig. 1 has been blocked out to show the relative arrangements of machinery and the ultimate possibilities of such a research department.

Naturally the preceding program of building and equipment may be considerably varied to meet the requirements of a given textile plant, but the principle involved and the general recommendations should be carried out in order to make the building and its contents fully adaptable to the purposes of a research laboratory. Frequently an unused portion of the plant itself may be utilized to house the laboratory, and again many plants have at their disposal the necessities of the laboratories regarding electrical currents, compressed air, heating and other items. This will of course make a substantial reduction in the expense.

When such a laboratory is first installed there will frequently arise a spirit of hesitancy or suspicion in the minds of the operating force for fear that such a laboratory will submit their products to a very rigid and undesirable inspection. This spirit, however, will very promptly change to one of complete cooperation and dependence when it is found that the laboratory staff are in thorough sympathy with the manufacturing departments and will assist to the utmost of their ability in the solution of actual difficulties. The manufacturing departments will then call frequently upon the laboratory staff to witness every difficulty in machine operation or production and will rely absolutely upon the word of the laboratory force for the solution of such questions.

The accompanying illustrations, Figs. 2 and 3, show the actual working out of such laboratories, designed and equipped by the author for the Albany Felt Co., Albany, N. Y. Fig. 2 illustrates the interior of the textile laboratory with its complement of testing machines, racks, etc. Fig. 3 shows similarly the chemical laboratory and coal room. These laboratories have been in operation about two years and have solved many hundreds of questions relative to the manufacture of textiles of the character of felts. These illustrations are inserted by courtesy of the Albany Felt Co.

THE PURCHASE OF MATERIALS ON SPECIFICATION

By Dean Harvey[1]

Synopsis

The increase in the demands on materials has created a need for a more exact statement of the requirements and, in many cases, improvement in quality. The most effective means of accomplishing this result is standardization. The purchase specification is an essential part of the program of standardization.

Some advantages of standardization, as brought out in this paper, are:

1. Simplification—reduction in number of materials used;
2. Uniform interpretation of requirements;
3. Uniform methods of sampling and testing;
4. Competitive bids from producers;
5. Better deliveries;
6. Stabilized production;
7. Facilitation of sales;
8. More knowledge of the properties of materials.

The advantages to be gained by the use of standard specifications are realized to the greatest extent when the specification is standard throughout the country. The A.S.T.M. has been the leader in the development of standard specifications and its standards should be used whenever practicable.

The procedure followed by a large manufacturing company in preparing specifications is described.

Some of the larger companies have established what is in effect a Materials Engineering Department having control of the quality of materials entering into apparatus. Its functions are to prepare purchase specifications, note performance of materials and modify requirements when necessary, also to gather data regarding the various materials used.

Under certain conditions the use of specifications may not be practicable. In order to be effective specifications must be backed up by inspection. The degree of inspection required depends upon circumstances.

The fundamental requirements for efficient purchasing have been stated as:

First: Specifying the right material.

Second: Buying the material desired on the best terms as to price and delivery.

Third: Insuring that the material received is as specified.

The use of complete or standard specifications will assist materially in carrying out these three requirements. By the term "specification,"

[1] Materials Engineer, Westinghouse Electric and Manufacturing Co., East Pittsburgh, Pa.

as used above, is meant a description of the quality of a material, used as the basis of purchase, giving the requirements in sufficient detail to insure the delivery of material of the desired quality.

Fifty years ago, very little attention was paid to specifications in purchasing materials. Materials were purchased only on general classifications. The consumer was somewhat familiar with the properties of the relatively small number of materials that he used. Since that time the enormous development of industry has caused a corresponding increase in the development of materials and many materials have been produced to meet the new needs which have arisen. Every year greater demands are being made upon materials than ever before. Industry is constantly calling for greater efficiency of equipment and this involves materials of superior properties or lower costs, or both. In order to be able to guarantee the performance of apparatus, manufacturers have been obliged to specify definitely the properties of the materials used. The need for a more exact statement of the requirements and, in many cases, improvement in quality, has been recognized. The most effective means of accomplishing these results is standardization of specifications.

As Warwick has so well stated,[1] "In its essentials, standardization is simply a process of selection of types, designs, materials or practices that in the course of time have thoroughly proved their value to the general community—a 'survival of the fittest'—and a concentration upon these types and materials in production and use in the interests of greatest efficiency." The reduction in the number of kinds of materials with a corresponding emphasis put on fewer products, each the most suitable of its kind, permits manufacture regularly and in relatively large quantities, resulting in improvement in quality and lower cost. The purchase specification is the medium through which the standard is expressed and put into use, and as such is an essential part of the program of standardization.

Some of the advantages to be gained by the use of standard specifications are:

1. *Simplification.*—One of the principal functions of standardization is the reduction in the number of materials used. When no standard is available, there gradually develops a large number of materials of the same general properties, but differing in minor details and requiring separate manufacture. This constitutes a large economic waste. Simplification, by concentrating attention on one

[1] C. L. Warwick, "Standard Specifications and Methods of Test for Materials," a paper presented at the First Pan-American Conference on Uniformity of Specifications, Lima, Peru, December 23, 1924.

standard material of a kind, permits production in larger quantities at lower cost. Better facilities for manufacture, inspection and testing can be provided, thus increasing the output and improving the quality and uniformity of the material. Smaller investment in the manufacturing equipment and lower stocks of raw materials and finished materials are the result.

The better quality and greater uniformity of standard materials may allow less expensive material to be used with a saving in cost.

2. *Interpretation of Requirements.*—Purchase specifications have the important function of giving all of the interested persons the same understanding of the requirements to be met. The purchasing agent uses a specification to serve as complete information to accompany a purchase order. He thus avoids the practice so often followed of giving a brief incomplete statement with an order which may be misunderstood and which also may be different for each order. The producer receives sufficient information for the manufacture. The inspector uses it as a guide in his inspection to determine the properties required and also the detailed methods of sampling and testing that are given, avoiding either too lax or too stringent inspection. The designer learns from it what properties he may expect in the material, aiding him in making the best application of materials in his designs. Without such a common source of information, misunderstandings are very likely to occur, and materials may be furnished which are not the best adapted for the purpose desired.

3. *Test Methods.*—Standard methods of sampling and testing enable both producer and consumer to test the material in the same way and obtain comparative results. The values determined for the properties of the materials often depend largely upon the manner of sampling and testing.

4. *Competitive Bids.*—By the use of specifications, the consumer obtains bids on the same quality of material from all producers. When there is no specification both quality and price quoted are likely to vary widely so that the purchaser is unable to make an intelligent comparison. Specifications are therefore a protection to the reputable manufacturer, who otherwise might be in competition with manufacturers of inferior products without the differences in quality being apparent.

5. *Deliveries.*—Better deliveries may be obtained by the consumer, as standard materials are usually more available than special products. In many cases they are carried in stock by the producer. Quicker deliveries warrant lower investment in stocks.

6. *Production.*—Production is stabilized, as there is a steadier

demand for standard than for special products. Standardization also allows manufacture for stock during periods of business depression. This keeps the plant running, thus reducing the labor turnover and keeping down overhead expense.

7. *Sales.*—Sales of materials are facilitated as established standards become widely known, and their merits are recognized.

8. *Knowledge of Materials.*—More complete and accurate information is available regarding the properties of materials bought on standard specifications. This is of much value to the designer, enabling him to apply the material more effectively.

The advantages obtained by the use of specifications that are recognized as standard throughout the country are, of course, much greater than with specifications that are standard only within the plant of a producer or a consumer. Materials so purchased are generally accepted as the best of their class. They are made in larger quantities, generally at lower cost, with more sources of supply and more markets than other materials. It should accordingly be our object to work toward national standard specifications and to use such standards whenever practicable.

The preparation of standard specifications for materials and methods of testing has been one of the principal objects of the American Society for Testing Materials since its organization in 1902. The organization of the Society, with its standing committees each composed of representatives of producers, consumers and independent interests, has rendered it particularly well adapted for this work. In this scheme of organization it is recognized that in order to obtain standard specifications of the greatest value to the industry concerned, it is necessary to have the cooperation of the producer to state the possibilities and limitations of manufacture and of the consumer to set forth the requirements for the application and to report the performance in service. By such cooperation the most suitable quality of material is obtained under favorable manufacturing conditions and prices.

A large portion of the 220 standard and 185 tentative specifications and methods of testing issued by the A.S.T.M. and which are now in force have been widely accepted as standard throughout the country.

The large number of national technical societies and trade organizations, each interested to a certain extent in the standardization, has made it essential that some arrangement be made for cooperation in standardization work. The organization of the American Engineering Standards Committee in 1918 provided a clearing house for

industrial standardization work. One of the objects of this committee is "to receive and pass upon recommendations for standards submitted to it by other organizations, in accordance with its Rules and Procedure with the hope of establishing American Engineering Standards; but not to formulate standards."

REQUIREMENTS FOR A SPECIFICATION

In addition to specifying the properties of the material desired, it is often necessary to include methods of sampling to insure representative samples, and methods of testing, because the values obtained for the properties mentioned are usually largely affected by the test methods employed. When, however, test methods have been established as standard practice in an industry, it becomes unnecessary to describe them in specifications. Any necessary instructions as to inspection should be given for the guidance both of the inspector and the producer. The specification also should give any desired instructions for packing and marking, and the procedure to be followed in connection with rejection of defective material.

One of the principal faults to be guarded against is making a specification too drastic. There should be the fewest possible restrictions consistent with obtaining the material required. Unnecessarily severe requirements react to the disadvantage of both the producer and the consumer, for they are likely to interfere with production, increase the cost and delay shipments. The statement sometimes made that "we use only the highest grade of materials" is often only a confession of ignorance. Needless expense may be incurred in this way when as good or better results may be obtained with a less expensive material. The properties of the material should be those that will best adapt it to the service. Higher quality is not necessary and is undesirable on account of increase in cost.

It is not necessary to dwell upon the fact that the requirements of a specification must be clearly set forth so as to avoid the possibility of misunderstandings. This demands clear statements, logically arranged. The standard form for specifications adopted by the A.S.T.M., with its logical arrangement, is excellent for this purpose. Brevity is a cardinal virtue in a specification, which should be as brief as possible consistent with clarity and completeness.

Both the producer and the consumer are interested primarily in the performance of the material in service. The emphasis should therefore be placed upon the performance rather than upon the composition, construction or method of manufacture. The producer should be given the greatest latitude in furnishing material for the

purpose desired. Where details of composition and manufacture are prescribed the producer is limited. He may be prevented from developing a product differing from that specified, but giving superior, or at least as good, performance at perhaps a lower price. In many cases it is not practicable to specify performance, and we are forced to describe the construction, but this should not be done when it can be avoided.

PREPARATION OF SPECIFICATIONS

It may be of interest to review briefly the procedure followed by the Westinghouse Electric and Manufacturing Co. in preparing purchase specifications. Two classes of specifications are used, known as complete and limited specifications. A complete specification is one in which the requirements for the material are fully given, allowing purchases to be made on the open market. A limited specification is one which gives only part of the requirements, making it necessary for the buyer to deal only with those producers who have furnished satisfactory material and understand what is needed.

There are many materials for which existing methods of test are not adequate to determine the essential characteristics. When a material that falls in this class is to be bought, the subject is discussed with the supplier, describing the application and such properties as are known to be requisites. A sample is obtained when practicable and given a laboratory test. A small trial order is then followed through the shop and the performance of the material is determined in the application for which it is intended. If satisfactory results are obtained the material is approved, and the producer is notified to furnish the same quality on future orders. A specification is prepared including such properties and test methods as are applicable, and purchases are limited to such suppliers as have furnished satisfactory trial orders. Another case in which the specification is limited, is a new material for which sufficient data has not yet been obtained to determine all of the essential properties. Sometimes such a material is needed before the data are available. Purchases are restricted to producers whose product has been found satisfactory and work on the specification is continued until the requirements are complete. A few specifications are confined to approved producers for certain materials difficult to manufacture, which it is desirable to obtain only from producers who are known to be able to furnish them, even though the specification gives a complete description. In all of these cases, limited specifications are used to advantage, in that they allow shipments to be checked even though all of the requirements are not given.

The same general procedure is followed in preparing a complete

specification as in the case of the limited specification. Unless it is known that material of certain properties will be suitable, a trial shipment is obtained before issuing the specification and allowing orders to be placed in the routine manner.

When selecting a new material, the specifications of the A.S.T.M. are reviewed and one of the standard grades of material is used whenever possible.

In the preparation of specifications, the first step is to decide upon the properties of the material desired and the method of determining these properties. Care is taken not to make the specification too cumbersome by adding unnecessary requirements. The specification is written and submitted for comment to the design engineer responsible for the application of the material, to the department of the shop where it will be used, and to the Inspection Department. It is then forwarded to the producer for his approval or criticism. The comments from these various sources are considered, the preliminary draft is modified if necessary, and the final specification is issued.

It is important to submit a new specification to the producer for his comments and obtain his approval before issuing it. He will tell whether he can produce material to meet the specification, and if not, what he can furnish. He can often give valuable suggestions for minor changes that will not interfere with the use of the material and will allow commercial products to be furnished instead of special grades. Also, the specifications will be accepted without question when submitted in connection with orders.

For many years we have been using the standard forms for specifications including abbreviations and nomenclature adopted by the A.S.T.M. and have found them admirably adapted to our needs. The standard specifications of the Society are used verbatim as far as practicable, adding the requirements to meet our local needs, such, for instance, as methods of packing and marking. They express the requirements clearly and concisely. They also facilitate purchasing as they already have been accepted by the industry. We have recently adopted the practice of making reference to the corresponding specification of the A.S.T.M. in the introductory paragraph, thus indicating that our specification calls for a standard product and incidentally giving publicity to the specifications of the Society.

Owing to the lack of knowledge of the properties of many materials when they are first used, and also to the undeveloped state of the art as regards methods of test for many products, a considerable proportion of specifications for materials is incomplete in some respects. Work is continually being carried on to extend our knowledge of

materials and improve methods of testing. It is also necessary to keep abreast of the times. A specification which reflected the best practice at the time it was written may be inadequate after a few years due to improvements in manufacture or to the development of a new material providing more desirable properties than those called for.

It is usually undesirable to include in a specification, requirements which cannot be checked by the purchaser, but there may be cases where such statements are of value in giving the producer a better idea of the materials desired.

Some of the larger companies have established what is in effect a Materials Engineering Department having control of the quality of materials entering into their products. Its functions include:

1. Preparing purchase specifications and keeping them up to date.
2. Noting the performance of the materials used and modifying the requirements when necessary.
3. Gathering data regarding the various classes of materials used. This includes the properties and methods of testing, new developments in materials and in general any information that would be of assistance to the designer. This work is independent of that of the Inspection Department and Testing Laboratory, which carry on the routine inspection and testing of incoming shipments. A Materials Department of this sort works closely with the Purchasing and Inspection Departments. It makes a study of materials and offers valuable assistance to the designers in using materials most effectively. It should be practicable for any company to assign at least one man to serve as the Materials Department. The results obtained have fully justified such an arrangement.

APPLICATION

It is sometimes claimed that the use of specifications increases prices. While there may sometimes be a tendency toward higher prices, particularly when the producer is not familiar with the specification, in general a quality specification, by permitting of competitive bidding, tends to lower prices rather than to raise them. The cost of preparing specifications and of inspecting materials is more than offset by the savings effected by the better quality and greater uniformity of the materials, and by the more effective use made possible through greater knowledge. In many cases the expense entailed in a few shipments of defective material would amount to as much as the cost of specifications and inspection for a long period of time.

Under certain conditions, the use of specifications for purchasing materials is not practicable. When a new material is first being used,

there may not be sufficient information regarding its properties and uniformity to allow a specification to be prepared, and it may be necessary to buy from approved producers until the necessary data have been accumulated. Or, specifications may not be practicable pending the development of adequate methods of test. Various commercial grades of materials may be sufficiently uniform in quality always to be suitable for certain uses where the requirements are easily met and it would therefore not be necessary to buy on specification. In many such cases the reason for the uniform quality will be found to be previous standardization of the material and the adoption of the standard specification by the producers.

In general, however, specifications are desirable for the purchase of materials. The benefits from their use have been fully recognized and appreciated both by producers and consumers.

ENFORCEMENT

It has been stated that specifications without inspection correspond to a code of laws without police power. While a specification alone offers the advantage of telling the producer what is desired, the omission of inspection lets down the bars to inferior products from careless or unscrupulous producers. It is therefore a handicap to the reputable manufacturer who cannot compete in price with competitors who quote very low prices and furnish material below grade.

In every well-regulated community the police supervision differs considerably in the various districts, being adapted according to the needs. Similarly the amount of inspection should be suited to the needs of the various materials. Certain important materials will always require most thorough inspection. New materials, or those obtained from new producers, will receive closer attention than the same products after they have become established. Commercial grades, bought on standard specifications from reliable manufacturers will not require such rigid inspection.

A number of testing companies throughout the country have facilities for inspection of materials for the purchaser at the plant of the producer. It is often desirable to utilize their services.

It has not been possible in the limited space available to do more than to state briefly some of the factors involved in the use of specifications for purchasing materials. Much valuable information on this subject has been given by Charles B. Dudley, in his annual addresses as president of the Society in 1903, 1904 and 1907, and by C. L. Warwick in his paper on "Standard Specifications and Methods of Test for Materials," presented at the First Pan-American Conference on Uniformity of Specifications, in Lima, Peru, December 23, 1924.

DISCUSSION

Mr. Gushee. MR. E. T. GUSHEE.[1]—Purchasing to correct specification is a subject which, from the very nature of my duties as purchasing agent, interests me deeply. As a purchasing agent I am at a loss to know what to buy, unless it is properly specified. Having purchased the material I am at a loss to know how to inspect it unless I have a specification as my guide.

If a purchasing agent has properly analyzed the requirements of his position, he must, if it is necessary, constitute himself the "driving force" in his organization to obtain a correct and complete specification file.

When a correct specification is first prepared, it has been my experience in many cases that the people using the material actually realize for the first time what performance they want from a given material. It is impossible to draw a careful specification without thorough analysis of the use of the material to be specified. Having accomplished this, it often happens that a better quality of material than has been purchased heretofore is needed—often that a quality not so good will suffice. It is the same with a guarantee. If you prepare a guarantee covering equipment you will learn what you actually require of that equipment. In this a guarantee is analogous to a specification.

I thoroughly agree with Mr. Harvey's conclusion that a quality specification tends to lower rather than to increase cost—primarily because it permits of competition. If the material or equipment under consideration is such that price is no object—a specification at least gives a basis of understanding between purchaser and vendor of what is required.

You may have noted my use of the phrase "correct specification." I have repeated the word "correct" purposely, for no specification at all is often better than one which is not correct. For that reason —and for others not necessary to detail here—I heartily concur with Mr. Harvey in that national standard specifications should be used, when possible, for, in general, these national specifications have been given the most careful consideration from every angle both by purchasers and vendors.

[1] Purchasing Agent, The Detroit Edison Co., Detroit, Mich.

Mr. Gushee.

There are two phases of the subject of specification, which I particularly wish to mention. First, the necessity of routine revision of specifications. A routine revision keeps the specification up to date and parallel to the advance of the art. If it is not routine it is generally lost sight of. Our specifications are revised or rather considered for revision every six months and no vendor is permitted to use a specification which does not show proper record of revision. Second, the advisability of permitting a vendor to quote you on his standard material, though it be at variance with your own specification. Our specifications bear the following notation:

"In addition to quoting on the material herein specified, the vendor is requested to submit a quotation on his standard material of similar characteristics."

I want to give an extract from a paper prepared by Mr. Hirshfeld and myself and delivered at a meeting of the Association of Edison Illuminating Companies at New London last year. It really sizes up the whole question of inspection and specification from the commercial aspect:

"If the executives here present will investigate the purchases made by their own companies most of them will undoubtedly be surprised to discover how little many of the requisitioning departments really know about the articles or materials that they ask to have purchased. Under such conditions who can blame a purchasing agent for buying exclusively on a price and delivery basis? Is it his fault if cleaning compounds and soaps damage the materials or structures on which they are used? Is it his fault if writing ink fades a few months after use or corrodes steel pens rapidly? Is it his fault if he purchases weatherproof wire containing jute instead of cotton in its covering? Is it his fault if threaded pipe fittings are so erratically threaded that several must be tried before one can be found to fit a given pipe? Most decidedly not. No man can purchase intelligently and to the best advantage until you can tell him what you want in such exact terms that he can tell the vendor exactly what is wanted and that an inspector can check the quality of the material delivered.

"Establishment of an inspection division in the case here under discussion led almost immediately to a realization of two facts. First, specifications for many items of purchase were lacking entirely, and second, the information on which specifications could be based was also lacking in many cases. The elimination of these deficiencies has occupied the time of several men ever since the discovery was made several years ago and the job is still far from finished.

"The preparation of such specifications is a cooperative proceeding, the inspection division furnishing the driving force on the plea that it cannot inspect without a proper definition of the material which is desired. When necessary it furnishes most or all of the technical knowledge and skill required to prepare proper and adequate specifications. Every effort is made to utilize specifications already in existence and when possible to utilize specifications prepared or adopted by a body of national scope or standing. Thus A.S.T.M. specifications

Mr. Gushee. and N.E.L.A. specifications are adopted and used whenever available and satisfactory. The formation of specifications on a national basis is now progressing rapidly in this country and it is intended to adopt these as soon as they become available and can be proved satisfactory."

I want again to emphasize the necessity, from the viewpoint of both purchaser and vendor, of the standardization of correct specifications. It is essential, and the Society is, of course, to be highly congratulated on the splendid work which it has done and is doing in this field.

Mr. Steele. MR. C. H. STEELE.[1]—There are just one or two devices that might be of interest to those who are working as specifications engineers. One is the matter of keeping a record. A convenient method is to use a large ledger sheet, perhaps 20 by 18 in., conveniently ruled in columns, with a space on the left for the manufacturer's name and the date and order number, using one such page for each specification. Let us assume a very simple one, like a chemical where perhaps you designate that the material itself must have, we will say, 98-per-cent purity; one specific impurity must not run over 1 per cent, and another impurity must not run over 0.5 per cent. Now, if you buy, as every good purchasing agent will buy, from more than one company, the convenient way is to list a shipment from manufacturer X when you get the report from the inspection department and from the laboratory, and note the fact that X furnished, say, 98.5-per-cent purity, and that he ran only 0.75 instead of 1 per cent in the first impurity, 0.3 per cent of the second impurity. You buy the next time perhaps from Y and get only possibly 97.5-per-cent purity, you may get only 0.5 per cent in the first impurity and 0.2 per cent in the second.

After you have kept the record for a year or more, you begin to get some very curious and interesting data. To begin with, you have in effect a birdseye view of all your purchases in a very convenient form, and a very simple method is to enter anything that falls below your designated specification in red ink. A glance will show you just who falls down and over a period of time you can weed out undesirable vendors. It is very convenient in tabulating average results of tests and noting maximum variations.

Your specifications do not stand still, for the reason that the arts are constantly improving, and you find after a while that possibly one manufacturer, through improved processes, has tended gradually to decrease certain impurities or eliminate them. In that case the logical thing to do is to revise your specification.

[1] Specification Division of Purchasing Department, Eastman Kodak Co., Rochester, N. Y.

Another convenience is to write a standard practicing instruction sheet for each individual who is concerned. That is, write an instruction sheet numbered with the same numbers as the specification, sending a copy to the laboratory who does the testing, the inspector who makes the preliminary inspections, and to the department that uses the material, stating all the things that everybody is to do and who is to be responsible for them. For example, your inspector will be held responsible for such things as packing, proper marking and notation of order numbers on the goods to save confusion in your stock department. Your laboratory naturally would be responsible for the physical or chemical tests. The department that uses the material sometimes is the best judge of certain things that can not be tested. For example, the printing surface on paper. Mr. Steele.

Another thing is to insure proper sampling. For example, your inspector or your receiving department—the first to come in contact with the material—is the logical man to take your sample. You as chemists and engineers know that sampling is two-thirds of the whole method of testing. If you do not get a representative sample your test does not mean anything. It is well in that instruction sheet to definitely instruct the person who does the sampling just what he shall take as a sample and how he shall take it.

One more thing that was brought to my mind by the last speaker is the matter of keeping specifications up to date. A very simple method that I know is being used is to number the specification, we will say No. 231, when it is written. If at any time it is revised, the revised edition is noted as 231*a;* if it is again revised the next revision is 231*b*, and so on. If you specify on the face of your order the fact that this order is placed on Specification 231*b*, you are protected on the point of the manufacturer using your most recently revised specification. If the order is placed with someone familiar with your product, you need not send a copy along with every order; but using a letter as an indication of the degree of revision keeps constantly in the mind of the manufacturer the fact that he has to use a certain edition of the specification.

MR. L. E. KERN.[1]—The A.S.T.M. has a member, the American Institute of Architects, whose members last year purchased between 2000 and 3000 different classes of articles at a total monetary value approximating two billion dollars. They purchased practically every single one of these articles on specification. It would seem, therefore, that the architect, as a purchasing agent, should be considered in Mr. Kern.

[1] Technical Secretary, Scientific Research Department, American Institute of Architects, New York City.

Mr. Kern. connection with the A.S.T.M. standard specifications for building materials, and that he should be very much interested in these standards. It is an unfortunate fact, however, that but very few of the A.S.T.M. standards are in general use among architects. There must be a reason for this. It is easier to use somebody else's specification than it is to get up one of your own. I have not prepared any discussion of this subject, but note that we have with us to-night a former Secretary of the American Institute of Architects, who is an authority on the subject of specifications, Mr. D. K. Boyd, and I should be very glad to yield the floor to him.

Mr. Boyd. MR. D. K. BOYD.[1]—This is an unexpected opportunity. As an architect and a member of this Society for a great many years, I have been much concerned about this subject, and believe that it is largely due to modesty on the part of the American Society for Testing Materials. I feel that with the very valuable specifications which the Society has relating to building materials that they should be made better known to the architectural profession. I have been advocating for a long time that each year there should be some sort of a document sent out to the members of the American Institute of Architects, at least, and preferably to the architects of the country, acquainting them with the specifications of this Society which apply to the materials used in construction. I have even suggested that a list should also be prepared and inserted in the front of the publication known as "Sweet's Catalog," which is widely used by architects. If there was a public registration, so to speak, of the specifications of the Society relating to buildings where architects could see just what materials are covered, I believe these specifications would be very widely used.

It was my pleasure yesterday to give a talk on brick, and I pointed out in connection with the specification for Building Brick of this Society what would probably apply to some others, and that is, the necessity for perhaps a reasonable modification of specifications for certain of the basic building materials which would make them more locally applicable. I think it is a fact that the specifications of this Society for steel and for cement, which can be produced at the mills and delivered all over the country with transportation an accepted part of the transaction, are, perhaps, quite familiar to architects and already extensively used.

In closing my brief part in this discussion, I do feel that the American Society for Testing Materials has not had a close enough contact with the architectural profession. As I said, perhaps it has

[1] Structural Standardist, Structural Service Bureau, Philadelphia.

been too modest, which again may have been due to lack of funds, but in any event I trust the specifications applicable to building materials will be made better known to architects generally. I thank you. **Mr. Boyd.**

THE CHAIRMAN (*Mr. J. A. Capp*).[1]—I believe the Chair is not supposed to discuss papers, but perhaps you can bear with me for a moment on just one matter which may be of some interest in connection particularly with the suggestion by Mr. Steele in regard to the numbering or designation of his specifications. In the work which it has fallen to my lot to do, we have found it necessary to follow essentially the same scheme that he has mentioned, that is, to give a permanent number to a specification, because that specification may be referred to on many drawings and in many other types of more or less permanent record. But specifications are not fixed, they are frequently revised, and we have, therefore, adopted the policy of always following the serial number, as we call it, of the specification with a number following a dash, which we call the designating number. We preface each specification with a cautionary note that all specifications covering the specified material will be issued under the mentioned serial number to be followed by a designating number, which is a complete reference to the particular issue; that it will be understood that on drawings and similar permanent records the serial number alone will appear, but that in correspondence, and especially in orders, the designating number must also appear in order that the reference may be complete. We caution vendors that if the designating number is omitted they must look on the reference as incomplete and come back to the order clerk to find out what specification in that series he means. We have found the system to work out very well because there is always a designating number following the serial number. If we simply started with 231 and then afterwards called it 231 *A*, *B*, and *C*, it would not work so well, but starting with 231–1, the first revision dash 2, the second dash 3, and so on, the vendor soon realizes that the dash number means as much to him as the main serial number. **Mr. Capp.**

PRESIDENT W. H. FULWEILER (*by letter*).—It may be of interest as pertaining to the discussion of Mr. Harvey's paper, to refer briefly to some work that has been done in connection with the development of the Directory of Commodity Specifications recently issued by the U. S. Department of Commerce. As the representative of the Society on the Advisory Committee formed by Mr. Hoover to aid in the preparation of the Directory, I was impressed with the need of some discussion of how to use purchase specifications, in order that the Directory might be of the maximum usefulness, and more especially to **President Fulweiler.**

[1] Chief of Testing Laboratory, General Electric Co., Schenectady, N. Y.

President Fulweiler. purchasing agents. I suggested that a Treatise on the Development and Use of Specifications should be prepared for distribution with the Directory, and I understand such a document is now in course of preparation.

In a report to the Executive Committee on this matter, I included the following outline of such a treatise as it appeared to me it might be drawn up. A discussion of this kind should include such topics as development, interpretation, use, limitations and choice of specifications, and in the outline there are suggestions for elaboration of each of these topics.

OUTLINE OF TREATISE ON SPECIFICATIONS

Suggested as a Supplement to the Directory of Commodity Specifications, U. S. Department of Commerce.

1. *General Formation and Development of Specifications:*

(*a*) Statement of what specifications are.

(*b*) Statement of how specifications are formed. For example, in beginning material is purchased on brand or reputation; then the user may start to test the material and in doing so discovers differences in behavior of various shipments and relates these differences to performance of various shipments and in that way begins to formulate ideas of the kind of materials he wants for a given purpose. This leads to a statement of requirements that would develop into specifications.

The mechanism of developing standard specifications may be classified as follows:

1. Preparation by consumer or associations of consumers, *e. g.*, Federal Government and such associations as the National Electric Light Association and the American Railway Engineering Association.
2. Preparation by producers or organizations of producers, frequently developed in connection with sales promotion work.
3. Preparation in organizations where both groups are represented on an equal footing, *e. g.*, the A.S.T.M.

(*c*) Show how specifications develop as the knowledge of the material or product grows.

Specifications may be classified into two types: (1) "Primary" specifications, which specify properties that directly determine the suitability of the material or product for its intended use; (2) "secondary" specifications, requirements of which are indirect and do not in themselves necessarily determine the actual suitability of the material or product. These types of specifications should be illustrated by selecting two or three and showing how they have developed.

When the engineering knowledge of a given material or product is vague we are apt to have secondary or indirect type of specifications. As knowledge develops it becomes possible to write the primary or direct type of specifications.

2. *Interpretation of Specifications:*

(*a*) Importance of methods of test in their relation to interpretation of specifications. No specification is any better than the methods of testing available for its interpretation. Illustrations will be given.

(*b*) Some requirements of specifications are susceptible of absolute or precise determination by general methods, whereas others are simply empirical and the method of test used determines the actual limits specified. Illustrations of both types will be given. **President Fulweiler.**

3. *Use of Specifications:*

(*a*) What will specifications do for the consumer? It is proposed to discuss this subject mainly from the point of view that specifications permit of competitive purchase with assurance of delivery of the material ordered.

(*b*) A discussion will be included of the general advantages of standard specifications and simplified practice along lines that have been used in various places.

4. *Limitations of Specifications:*

(*a*) The purchaser must have a knowledge of what he really wants.

(*b*) Methods of test may place a very definite limitation on specifications.

(*c*) The purchaser must be prepared to enforce the specifications by adequate inspecting and testing; otherwise he cannot be sure that proper deliveries under the specifications are being made.

(*d*) As a possible alternative some plan of brand, certification or guarantee by the producer that his product meets the specifications might be considered.

(*e*) Limitations of specifications might be illustrated by mentioning certain ways in which not to use specifications.

5. *Choice of Specifications:*

In this section it is suggested that some statements be included as to how to select a specification from the Directory.

(*a*) The purchaser must first determine from the Directory, and, if necessary, from a comparison of the specifications themselves, those which seem to cover more nearly the type of material or product he desires.

In this connection national societies are glad to assist the user in their problems of this kind.

(*b*) The purchaser should keep in mind the various principles above developed, that is, the adequacy of methods of testing and features involved in the use and limitation of specifications.

(*c*) It should be urged that purchasers should not write their own specifications by the "scissors and paste pot method."

VOLUME 25, PART II

SUBJECT INDEX

A

Abrasion.
Mortar abrasion test, 223.
The Significance of the Common Test Methods for Determining the Strength of Mortars. J. W. Gowen and H. W. Leavitt, 218. Discussion, 228.

Absorption.
Brick. D. K. Boyd, 315. Discussion, 326.

Aggregates.
Analytical Properties of Set and Hardened Mortars. E. E. Butterfield, 283. Discussion, 292.
A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures. Prévost Hubbard and F. C. Field, 335.

Alloys.
A Note on the Microstructure of Aluminum-Iron Alloys of High Purity. E. H. Dix, Jr., 120. Discussion, 130.
Some Fatigue Tests on Non-Ferrous Metals. R. R. Moore, 66. Discussion, 84.

Alloy Steel.
Fatigue of Metals by Direct Stress. P. L. Irwin, 53. Discussion, 64.
The Tensile Properties of Metals at High Temperatures. T. D. Lynch, N. L. Mochel and P. G. McVetty, 5.

Aluminum.
Accelerated Corrosion Tests on Bare Overhead Electrical Conductors. F. F. Fowle, 137. Discussion, 153.
Comparative corrosion data on aluminum alloys, 166.
A Note on the Microstructure of Aluminum-Iron Alloys of High Purity. E. H. Dix, Jr., 120. Discussion, 130.
Physical properties of non-ferrous metals, 69.
Some Fatigue Tests on Non-Ferrous Metals. R. R. Moore, 66. Discussion, 84.

Asphalt.
A Deformation Test for Asphaltic Mixtures. H. M. Milburn, 356.
A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures. Prévost Hubbard and F. C. Field, 335.
A Stability Test for Bituminous Paving Mixtures. W. J. Emmons and B. A. Anderton, 346.

B

Bituminous Materials.
A Deformation Test for Asphaltic Mixtures. H. M. Milburn, 356.
Discussion on stability of bituminous paving mixtures, 365.

Effect of filler on stability of natural sand mixtures with 10 per cent of added asphalt, 343.
Machine for the Determination of the Pliability of Prepared Roofing and the Breaking Point of Bitumen. C. S. Reeve and F. W. Yeager, 385. Discussion, 390.
A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures. Prévost Hubbard and F. C. Field, 335.
Quality of Oil for Surface Oiling of Earth Roads and Streets. F. L. Sperry, 376.
Relation between voids and strength; gravel asphaltic concrete, 355.
Relation between voids and strength with changes in bitumen; sheet asphalt, 352.
Relation between voids and strength with changes in bitumen; stone asphaltic concrete, 354.
Relation between voids and strength with changes in filler; sheet asphalt, 353.
A Stability Test for Bituminous Paving Mixtures. W. J. Emmons and B. A. Anderton, 346.

Brass.

Physical properties of non-ferrous metals, 69.
Some Fatigue Tests on Non-Ferrous Metals. R. R. Moore, 66. Discussion, 84.

Brick.

Brick. D. K. Boyd, 315. Discussion, 326.
Influence of the strength of mortar on piers, 324.

Bronze.

Fatigue of Metals by Direct Stress. P. L. Irwin, 53. Discussion, 64.
The Tensile Properties of Metals at High Temperatures. T. D. Lynch, N. L. Mochel and P. G. McVetty, 5.

C

Cast Iron.

Typical Static and Fatigue Tests on Steel at Elevated Temperatures. T. McL. Jasper, 27.

Cement.

Comparative tests of high-alumina and portland cements, 193.
Discussion on cement testing, 177.
Requirements of Cement for Modern Highway Construction. A. T. Goldbeck, 169. Discussion, 177.
A Study of Temperatures in High-Alumina Cement and Methods of Curing. H. S. Mattimore, 192. Discussion, 198.

Cold Working.

Softening of Hard-Rolled Electrolytic Copper. N. B. Pilling and G. P. Halliwell, 97.

Color.

Tests for Hardness, Gloss, Color and Leveling of Varnishes. A. H. Pfund, 392. Discussion, 403.

Compression Testing.

A Deformation Test for Asphaltic Mixtures. H. M. Milburn, 356.
Discussion on concrete compression test specimens, 251.
Discussion on stability of bituminous paving mixtures, 365.
Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete. H. F. Gonnerman, 237. Discussion, 251.
A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures. Prévost Hubbard and F. C. Field, 335.
Relation between tension, transverse and compression tests of mortars at various ages, 230.
The Significance of the Common Test Methods for Determining the Strength of Mortars. J. W. Gowen and H. W. Leavitt, 218. Discussion, 228.
A Stability Test for Bituminous Paving Mixtures. W. J. Emmons and B. A. Anderton, 346.

Concrete.

Analytical Properties of Set and Hardened Mortars. E. E. Butterfield, 283. Discussion, 292.
Chemical analysis of mortars, 286.
Comparison of flexural and compressive strength of concrete, 189.
Discussion on bond between concrete and steel, 273.
Discussion on concrete compression test specimens, 251.
Discussion on curing of concrete, 198.
Discussion on testing consistency of concrete, 213.
Effect of age on compressive strength of concrete, 247.
Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete. H. F. Gonnerman, 237. Discussion, 251.
Effect on bond of replacing cement with hydrated lime, 270.
A New Test for Consistency of Paving Concrete. F. H. Jackson and George Werner, 204. Discussion, 213.
Relation of voids in mortar to ratio of sand to cement for mortar of various water content, 293.
The Significance of the Common Test Methods for Determining the Strength of Mortars. J. W. Gowen and H. W. Leavitt, 218. Discussion, 228.
Studies of Bond Between Concrete and Steel. D. A. Abrams, 256. Discussion, 273.
A Study of Temperatures in High-Alumina Cement and Methods of Curing. H. S. Mattimore, 192. Discussion, 198.

Consistency.

Discussion on testing consistency of concrete, 213.
A New Test for Consistency of Paving Concrete. F. H. Jackson and George Werner, 204. Discussion, 213.

Copper.

Accelerated Corrosion Tests on Bare Overhead Electrical Conductors. F. F. Fowle, 137. Discussion, 153.
Softening of Hard-Rolled Electrolytic Copper. N. B. Pilling and G. P. Halliwell, 97.

Corrosion.
Accelerated Corrosion Tests on Bare Overhead Electrical Conductors. F. F. Fowle, 137. Discussion, 153.
Comparative corrosion data on aluminum alloys, 166.
Discussion on accelerated corrosion tests, 153.
Discussion on evaluation of corrosion tests, 161.
Effect of corrosion on physical properties, 158.
The Evaluation of Corrosion Tests. E. Blough, 156. Discussion, 161.

F

Fatigue Testing.
Comparison of effect of notch on endurance limit with elongation of material, 78.
Fatigue of Metals by Direct Stress. P. L. Irwin, 53. Discussion, 64.
References on fatigue testing, 83.
Some Fatigue Tests on Non-Ferrous Metals. R. R. Moore, 66. Discussion, 84.
Typical Static and Fatigue Tests on Steel at Elevated Temperatures. T. McL. Jasper, 27.

Fire Tests.
The Fire Resistance of Gypsum Partitions. S. H. Ingberg, 299.

Flexure Testing.
Comparison of flexural and compressive strength of concrete, 189.
Relation between tension, transverse and compression tests of mortars at various ages, 230.

G

Gloss.
Tests for Hardness, Gloss, Color and Leveling of Varnishes. A. H. Pfund, 392. Discussion, 403.

Gypsum.
The Fire Resistance of Gypsum Partitions. S. H. Ingberg, 299.
Strength, absorption and weight of gypsum block, 300.

H

Hardness.
Tests for Hardness, Gloss, Color and Leveling of Varnishes. A. H. Pfund, 392. Discussion, 403.

Heat Treatment.
Softening of Hard-Rolled Electrolytic Copper. N. B. Pilling and G. P. Halliwell, 97.

High Temperature.
Discussion on Properties of Metals at Elevated Temperatures, 33.
The Tensile Properties of Metals at High Temperatures. T. D. Lynch, N. L. Mochel and P. G. McVetty, 5.
Typical Static and Fatigue Tests on Steel at Elevated Temperatures. T. McL. Jasper, 27.

I

Iron.
Accelerated Corrosion Tests on Bare Overhead Electrical Conductors. F. F. Fowle, 137. Discussion, 153.
A Note on the Microstructure of Aluminum-Iron Alloys of High Purity. E. H. Dix, Jr., 120. Discussion, 130.

L

Laboratory.
The Design of a Research Laboratory for a Textile Manufacturing Plant. G. B. Haven, 416.
Lime.
Effect on bond of replacing cement with hydrated lime, 270.

Long-Time Tests.
Discussion on Properties of Metals at Elevated Temperatures, 33.
The Tensile Properties of Metals at High Temperatures. T. D. Lynch, N. L. Mochel and P. G. McVetty, 5.

M

Magnesium.
Physical properties of non-ferrous metals, 69.
Some Fatigue Tests on Non-Ferrous Metals. R. R. Moore, 66. Discussion, 84.

Metallography.
A Note on the Microstructure of Aluminum-Iron Alloys of High Purity. E. H. Dix, Jr., 120. Discussion, 130.

Mortar.
Analytical Properties of Set and Hardened Mortars. E. E. Butterfield, 283. Discussion, 292.
Chemical analysis of mortars, 286.
Influence of the strength of mortar on piers, 324.
Mortar abrasion test, 223.
Relation between tension, transverse and compression tests of mortars at various ages, 230.
Relation of voids in mortar to ratio of sand to cement for mortar of various water content, 293.
The Significance of the Common Test Methods for Determining the Strength of Mortars. J. W. Gowen and H. W. Leavitt, 218. Discussion, 228.

O

Oil.
Physical and chemical characteristics of road oils, 378.
Quality of Oil for Surface Oiling of Earth Roads and Streets. F. L. Sperry, 376.

P

Paint.
A Recording Drying-Time Meter for Varnishes and Similar Materials. J. McE. Sanderson, 407.
Tests for Hardness, Gloss, Color and Leveling of Varnishes. A. H. Pfund, 392. Discussion, 403.

R

Research.

The Design of a Research Laboratory for a Textile Manufacturing Plant. G. B. Haven, 416.

Road Material.

A Deformation Test for Asphaltic Mixtures. H. M. Milburn, 356.

Discussion on stability of bituminous paving mixtures, 365.

Effect of filler on stability of natural sand mixtures with 10 per cent of added asphalt, 343.

Effect of filler on stability of natural sand mixtures with 10 per cent of castor oil, 344.

A New Test for Consistency of Paving Concrete. F. H. Jackson and George Werner, 204. Discussion, 213.

Physical and chemical characteristics of road oils used in Illinois experiments, 378.

A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures. Prévost Hubbard and F. C. Field, 335.

Quality of Oil for Surface Oiling of Earth Roads and Streets. F. L. Sperry, 376.

Relation between voids and strength; gravel asphaltic concrete, 355.

Relation between voids and strength with changes in bitumen; sheet asphalt, 352.

Relation between voids and strength with changes in bitumen; stone asphaltic concrete, 354.

Relation between voids and strength with changes in filler; sheet asphalt, 353.

Requirements of Cement for Modern Highway Construction. A. T. Goldbeck, 169. Discussion, 177.

A Stability Test for Bituminous Paving Mixtures. W. J. Emmons and B. A. Anderton, 346.

A Study of Temperatures in High-Alumina Cement and Methods of Curing. H. S. Mattimore, 192. Discussion, 198.

Roofing.

Machine for the Determination of the Pliability of Prepared Roofing and the Breaking Point of Bitumen. C. S. Reeve and F. W. Yeager, 385. Discussion, 390.

S

Specifications.

Brick. D. K. Boyd, 315. Discussion, 326.

Outline of treatise on specifications, 440.

The Purchase of Materials on Specifications. Dean Harvey, 425. Discussion, 434.

Steel.

Accelerated Corrosion Tests on Bare Overhead Electrical Conductors. F. F. Fowle, 137. Discussion, 153.

Discussion on properties of metals at elevated temperatures, 33.

Fatigue of Metals by Direct Stress. P. L. Irwin, 53. Discussion, 64.

Studies of Bond Between Concrete and Steel. D. A. Abrams, 256. Discussion, 273.

The Tensile Properties of Metals at High Temperatures. T. D. Lynch, N. L. Mochel and P. G. McVetty, 5.
Typical Static and Fatigue Tests on Steel at Elevated Temperatures. T. McL. Jasper, 27.

T

Testing Apparatus.
The Design of a Research Laboratory for a Textile Manufacturing Plant. G. B. Haven, 416.
A Recording Drying-Time Meter for Varnishes and Similar Materials. J. McE. Sanderson, 407.
Tests for Hardness, Gloss, Color and Leveling of Varnishes. A. H. Pfund, 392. Discussion, 403.

Testing, Methods of.
Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete. H. F. Gonnerman, 237. Discussion, 251.
The Significance of the Common Test Methods for Determining the Strength of Mortars. J. W. Gowen and H. W. Leavitt, 218. Discussion, 228.

Textiles.
The Design of a Research Laboratory for a Textile Manufacturing Plant. G. B. Haven, 416.

V

Varnish.
A Recording Drying-Time Meter for Varnishes and Similar Materials. J. McE. Sanderson, 407.
Tests for Hardness, Gloss, Color and Leveling of Varnishes. A. H. Pfund, 392. Discussion, 403.

W

Wire.
Accelerated Corrosion Tests on Bare Overhead Electrical Conductors. F. F. Fowle, 137. Discussion, 153.

AUTHOR INDEX

A

Abrams, D. A.
Studies of Bond Between Concrete and Steel, 256.
Discussion, 185, 189, 279.

Adams, W. C.
Discussion, 390.

Anderton, B. A.
A Stability Test for Bituminous Paving Mixtures, 346.
Discussion, 373.

Ashton, Ernest.
Discussion, 214.

B

Blough, E.
The Evaluation of Corrosion Tests, 156.
Discussion, 162.

Boyd, D. K.
Brick, 315.
Discussion, 328, 334, 438.

Bragg, J. G.
Discussion, 177, 188.

Breyer, F. G.
Discussion, 153, 154, 164.

Butterfield, E. E.
Analytical Properties of Set and Hardened Mortars, 283.
Discussion, 251, 298.

C

Capp, J. A.
Discussion, 439.

Carver, William.
Discussion, 332.

Clemmer, H. F.
Discussion, 179.

Conwell, E. L.
Discussion, 198.

Crum, R. W.
Discussion, 273.

D

Dix, E. H., Jr.
A Note on the Microstructure of Aluminum-Iron Alloys of High Purity, 120.
Discussion, 131, 135.

E

Emmons, W. J.
A Stability Test for Bituminous Paving Mixtures, 346.

F

Field, F. C.
A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures, 335.

Fitch, T. A.
Discussion, 188, 375.

Fowle, F. F.
Accelerated Corrosion Tests on Bare Overhead Electrical Conductors, 137.
Discussion, 155.

Freeman, P. J.
Discussion, 202.

French, H. J.
Discussion, 34.

Fulweiler, W. H.
Discussion, 439.

G

Gilkey, H. J.
Discussion, 214.

Goldbeck, A. T.
Requirements of Cement for Modern Highway Construction, 169.
Discussion, 189.

Gonnerman, H. F.
Effect of Size and Shape of Test Specimen on Compressive Strength of Concrete, 237.
Discussion, 253.

Gowen, J. W.
The Significance of the Common Test Methods for Determining the Strength of Mortars, 218.
Discussion, 235.

Gushee, E. T.
Discussion, 434.

H

Halliwell, G. P.
Softening of Hard-Rolled Electrolytic Copper, 97.

Harvey, Dean.
The Purchase of Materials on Specification, 425.

Haven, G. B.
The Design of a Research Laboratory for a Textile Manufacturing Plant, 416.

Herschel, W. H.
Discussion, 216, 217.

Howe, H. L., Jr.
Discussion, 371.

Hubbard, Prévost.
A Practical Method for Determining the Relative Stability of Fine-Aggregate Asphalt Paving Mixtures, 335.
Discussion, 373.

Hutchinson, G. W.
Discussion, 230, 278.

I

Ingberg, S. H.
The Fire Resistance of Gypsum Partitions, 299.
Discussion, 214, 331.

Irwin, P. L.
Fatigue of Metals by Direct Stress, 53.
Discussion, 65.

J

Jackson, F. H.
A New Test for Consistency of Paving Concrete, 204.
Discussion, 214, 216.

Jasper, T. McL.
Typical Static and Fatigue Tests on Steel at Elevated Temperatures, 27.
Discussion, 41, 50.

Job, Robert.
Discussion, 162, 164.

Johnson, N. C.
Discussion, 185.

K

Kern, L. E.
Discussion, 437.

Koch, W. C.
Discussion, 327.

L

Lawson, T. R.
Discussion, 326, 330.

Leavitt, H. W.
The Significance of the Common Test Methods for Determining the Strength of Mortars, 218.
Discussion, 234, 235.

Levison, A. A.
Discussion, 215.

Little, W. F.
Discussion, 404.

Lynch, T. D.
The Tensile Properties of Metals at High Temperatures, 5.
Discussion, 40, 48.

M

MacKenzie, J. T.
Discussion, 162.

Malcolm, V. T.
Discussion, 43.

Mattimore, H. S.
A Study of Temperatures in High-Alumina Cement and Methods of Curing, 192.
Discussion, 181, 203.

McAdam, D. J.
Discussion, 84, 130, 132.

McKay, R. J.
Discussion, 154.

McKesson, C. L.
Discussion, 181.

McVetty, P. G.
The Tensile Properties of Metals at High Temperatures, 5.
Discussion, 48.

Merriman, Thaddeus.
Discussion, 183, 191, 201.

Milburn, H. M.
A Deformation Test for Asphaltic Mixtures, 356.

Miner, J. L.
Discussion, 200.

Mochel, N. L.
The Tensile Properties of Metals at High Temperatures, 5.
Discussion, 48.

Moore, H. F.
Discussion, 64.

Moore, R. R.
Some Fatigue Tests on Non-Ferrous Metals, 66.
Discussion, 90.

Myers, Bert.
Discussion, 273.

N

Nelson, H. A.
Discussion, 403.

P

Perkins, G. H.
Discussion, 371, 374.

Pfund, A. H.
Tests for Hardness, Gloss, Color and Leveling of Varnishes, 392.
Discussion, 403, 404.

Pierce, W. M.
Discussion, 163.

Pilling, N. B.
Softening of Hard-Rolled Electrolytic Copper, 97.
Proudley, C. E.
Discussion, 228.

R

Randall, T. A.
Discussion, 330.
Rawdon, H. S.
Discussion, 153, 163.
Reeve, C. S.
Machine for the Determination of the Pliability of Prepared Roofing and the Breaking Point of Bitumen, 385.
Discussion, 391.
Richart, F. E.
Discussion, 292.

S

Sanderson, J. McE.
A Recording Drying-Time Meter for Varnishes and Similar Materials, 407.
Sayre, M. F.
Discussion, 165.
Schmutz, F. C.
Discussion, 405.
Slater, W. A.
Discussion, 213, 216, 276.
Smith, E. B.
Discussion, 273.
Smith, F. P.
Discussion, 370, 375.
Speller, F. N.
Discussion, 162.
Sperry, F. L.
Quality of Oil for Surface Oiling of Earth Roads and Streets, 376.
Steele, C. H.
Discussion, 436.

T

Talbot, A. N.
Discussion, 332.
Templin, R. L.
Discussion, 95.
Thum, E. E.
Discussion, 161.

U

Ulman, M. H.
Discussion, 365, 373.

V

Viens, E.
Discussion, 332.

W

Werner, George.
A New Test for Consistency of Paving Concrete, 204.

Wig, R. J.
Discussion, 216.

Wood, J. K.
Discussion, 33.

Woolson, I. H.
Discussion, 328, 331.

Y

Yeager, F. W.
Machine for the Determination of the Pliability of Prepared Roofing and the Breaking Point of Bitumen, 385.
Discussion, 391.

Young, R. B.
Discussion, 191.